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PHASE I INSPECTION REPORT

NATIONAL DAM SAFETY PROGRAM

LONE TREE CREEK BASIN
VAUX #1 AND VAUX #2 DAMS
SIDNEY, MONTANA
MT-357 AND MT-358

PREPARED FOR:

HONORABLE THOMAS L. JUDGE
GOVERNOR, STATE OF MONTANA

LONE TREE RANCH CORPORATION
(OWNER AND OPERATOR)

PREPARED BY:

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CONSULTING ENGINEERS

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U.S. ARMY
CORPS OF ENGINEERS
Seattle
DISTRICT

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EXECUTIVE SUMMARY

VAUX #1 and VAUX #2 DAM

Morrison-Maierle, Inc. under contract to the Montana Department of Natural Resources and Conservation who are under contract to Seattle District, U.S. Army Corps of Engineers, inspected Vaux #1 and #2 Dams on September 27, 1978 under authority of Public Law 92-367 and in accordance with the "Recommended Guidelines for Safety Inspection of Dams" prepared by the U.S. Department of the Army, Office of the Chief of Engineers. The Vaux Dams are owned and operated by the Lone Tree Ranch Corporation and are located on Lone Tree Creek approximately three miles upstream of the City of Sidney, Montana in Richland County.

FINDINGS AND EVALUATIONS.

Vaux #1 Dam and Vaux #2 Dam are two separate earthfill dams each creating its own impoundment but operated jointly as one irrigation water supply project. Vaux #2 is the larger of the two dams situated about one-half mile upstream of Vaux #1. Vaux #2 Dam is 46 feet high and impounds 600 acre-feet of water at normal pool and 1175 acre-feet at the crest of the dam. It functions as the project's main storage reservoir and also has the only project spillway. The lower dam, Vaux #1, is 24 feet high, impounds approximately 250 acre-feet at normal pool and 685 acre-feet at the dam crest. It is a storage and regulating reservoir which controls all project irrigation releases. Vaux #2 controls essentially the entire inflow to the Vaux #1 Reservoir. A spillway which once served the lower dam has been completely obstructed by the construction of two roads across the channel, and under present conditions, Vaux #1 has no functional spillway.

Vaux #1 Dam was constructed in 1936 and Vaux #2 Dam was constructed in 1944 and raised in 1947. In March 1951 an ice jam in the spillway caused Vaux #2 to overtop and fail which in turn caused Vaux #1 to fail in a similar manner. The failure caused extensive property damage downstream in the City of Sidney but there was no loss of life. In 1960 both dams were rebuilt as they exist today.

Because both dams function as one project, this investigation combines both dams into one project classification. Inspection criteria classifies this project as intermediate in size and because of possible severe downstream effects of failure, the downstream hazard potential is high (category 1). Inspection criteria recommends that such projects safely handle a full Probable Maximum Flood (PMF). The PMF is the flood expected from the most severe combination of critical meteorologic and hydrologic conditions reasonably possible in the watershed. The PMF was assumed to occur in June as a result of the rainfall from a 72-hour storm. The PMF would cause a peak inflow to Vaux #2 Reservoir of 93,000 cubic feet per second (cfs) and a flood volume of 77,500 acre-feet.

Vaux #2 Reservoir was assumed to be at the spillway crest (80.9 feet project datum) at the beginning of the PMF. Because the outlet conduit has a relatively insignificant discharge capacity it was not considered in the analysis. The unlined open channel chute spillway was considered fully operational. Because of inadequate discharge and storage capacity, the reservoir would rise to the top of the dam (elevation 91.9 feet project datum) early in the flood, adding about 575 acre-feet of surcharge storage to the 600 acre-feet of normal storage at the spillway crest. At the time of overtopping, the total discharge would have increased to about 4,350 cfs through the spillway. Using this combination of discharge and surcharge storage the Vaux #2 Dam would handle only about 5% of the PMF volume.

Because of the nature of the dam embankment material, rapid failure of the structure would result from any overtopping, much as occured in 1951. Because Vaux #1 has no spillway and its reservoir capacity is less than Vaux #2, failure of Vaux #2 would cause overtopping and failure of Vaux #1 adding the volume of Vaux #1 Reservoir to the flood surge. Failure of the entire project would cause severe downstream flooding, property destruction and the potential for loss of life as the flood surge traveled down the valley into the town of Sidney.

The on-site inspection of the dams revealed wave cut benches in both dams. In addition a free-flowing seepage was observed at both dams which suggests that downstream slopes may not meet minimum stability criteria. However, there was no evidence of resulting piping or slope failure in the embankments. The greatest seepage occurs immediately downstream of the south abutment of Vaux #2 where a "quivering ground" condition was observed at the surface. Also, the unlined spillway channel is highly eroded beginning about 1300 feet from the entrance where a 37 foot deep gully has been cut in the channel. Stability of the Vaux #2 embankment is not affected at this time but could be if the headward erosion progresses another 900 feet.

Because the project can safely handle only 5% of the PMF, without overtopping and causing the dam to fail, the Vaux dams are considered unsafe until recommended actions are completed. In addition, seepage and possible stability questions are of concern. Action to resolve all issues needs to be taken without delay.

RECOMMENDATIONS

- Immediately prepare, implement and test an emergency plan for alerting downstream residents in case of imminent dam overtopping.

- Immediately remove the roads blocking the spillway at Vaux #1 and restore the spillway to service.

- Until the Vaux #1 spillway is made operational, prepare and implement a project operating plan to assure the outlet discharge from Vaux #2 Dam does not exceed the discharge and/or storage capacity of Vaux #1 Dam to insure Vaux #1 Dam does not overtop during normal operating conditions.

- Prior to next refill period, accomplish remedial repair to the Vaux #2 Dam spillway channel to arrest the erosion that has occurred about 1300 feet downstream from the entrance.

- Repair eroded areas of upstream face of embankments and place adequate riprap protection.

- Conduct engineering studies and investigations to determine the PMF and to perform stability evaluations.

- As studies indicate, modify the project to safely handle the full PMF. If the project cannot be made safe within a reasonable period of time, remove the dams to protect downstream life and property.

Rodger C. Foster

Rodger C. Foster, P.E.
Project Manager

TABLE 1 - PERTINENT DATA

VAUX #1 DAM

1. GENERAL

Federal Identification No.	-	MT-357
Owner Operator	-	Lone Tree Ranch Corp.
Purpose	-	Irrigation storage and regulation.
Location	-	3 miles upstream of Sidney, Montana in Richland County. Township 23 North, Range 58 East, Section 25, Longitude 104° 13' 30", Latitude 47° 43' 40". USGS Sidney, Montana Quadrangle.
Watershed	-	Lone Tree Creek
Design and Construction	-	Originally constructed in 1936. Dam failed March 26, 1951. Repair designed by Lillis Engineering of Billings, MT. and constructed in 1960 by Buller Construction Company. Repair construction supervision provided by Burdick & Webster of Williston, N.D.
Size Classification	-	Small
Downstream Hazard Potential	-	High
Datum	-	All elevations to Vaux #2 project datum.

2. RESERVOIR

Surface Area	-	30 acres at normal pool elevation 47.3 feet.
Length of Normal Pool.	-	2200 Feet.
Storage at Normal Pool	-	250 AF at elevation 47.3 Feet.

TABLE 1 - PERTINENT DATA - Continued

VAUX #1 DAM

Surcharge Storage	-	435AF from normal pool to dam crest.
Maximum Storage	-	685AF at dam crest elev. 54.0 feet.
Drainage Area	-	83 Square Miles.

3. SPILLWAY (Abandoned)

Location	-	In North abutment adjacent to enbankment and cut through natural divide to Brorson Creek.
Type	-	Unlined open channel chute spillway.
Geometry	-	Trapezoidal channel with 100 ft. bottom width and a 1v on 2h side slopes. Total length of 235 feet.
Control	-	2 road fills cross the spill-way channel at min. elevation of 55.3 feet. Road prohibits spillway from functioning. exist.
Crest Elevation	-	48.9 feet maximum channel bed elevation.
Slope	-	.5%
Capacity	-	Unknown. As presently exists, capacity is zero.

4. OUTLET

Type	-	24 inch RC pipe through the embankment. Wet well gate shaft located at center of dam cross section.

TABLE 1 - PERTNENT DATA - Continued

VAUX #1 DAM

Control	-	Slide gate on exit from gate shaft.
Length	-	150 feet entrance to exit.
Capacity	-	50 cfs

5. DAM

Type	-	Earthfill.
Hydraulic Height	-	24 feet from crest to outlet.
Crest Elevation	-	54.0 Feet.
Crest Width	-	800 Feet.
Upstream Slope	-	Variable 1V on 2.5H to 1V on 3H.
Downstream Slope	-	Variable 1V on 2H to 1V on 2.5H.

TABLE 2 - PERTINENT DATA

VAUX #2 DAM

1. GENERAL

Federal Identification No.	-	MT-358
Owner/Operator	-	Lone Tree Ranch Coorporation
Purpose	-	Irrigation Storage.
Location	-	3 miles upstream of Sidney, Montana and .5 mile upstream of Vaux #1 Dam in Richland County. Township 23 North, Range 58 East, Section 26, Longitude 104° 14' 08", Latitude 47° 43' 48". USGS Sidney Montana Quadrangle.
Watershed	-	Lone Tree Creek
Design and Construction	-	Originally constructed in 1944 and raised in 1947. Dam failed March 26, 1951. Repair design by Lillis Engineering of Billings, MT and constructed in 1960 by Buller Construction Company. Repair construction supervised by Burdict & Webster of Williston, N.D.
Size Classification	-	Intermediate
Downstream Hazard Potential	-	High
Datum	-	All elevations to Vaux #2 project Datum.

2. RESERVOIR

Surface Area	-	50 Acres at normal pool (spillway crest).

TABLE 2 - PERTINENT DATA - Continued

VAUX #2 DAM

Length of Normal Pool	-	3200 Feet.
Storage at Normal Pool	-	600AF at spillway crest elevation 80.9 feet.
Surcharge Storage	-	575AF spillway crest to dam crest elevation 91.9 feet.
Maximum Storage	-	1175AF dam crest elevation 91.9 feet.
Drainage Area	-	83 square miles.

3. SPILLWAY

Location	-	400 feet upstream of north abutment separate from embankment discharges to Brorson Creek.
Type	-	Unlined open channel chute spillway.
Geometry	-	Trapezoidal channel with 45 pool bottom and side slopes of IV on 2H Total length of 2000 feet.
Slope	-	Flat to .3%.
Control	-	No structural control. Entrance to channel approximately 115 feet wide at bed.
Crest Elevation	-	80.9 feet maximum channel bed elevation.
Capacity	-	4350 cfs at overtopping elev. 91.9 feet.

4. OUTLET

Type	-	24 inch RC pipe through embankment. Wet well gate shaft located at center of dam cross section.

TABLE 2 - PERTINENT DATA - Continued

VAUX #2 DAM

Control	-	Slide gate on exit from gate shaft.
Length	-	270 feet entrance to exit.
Capacity	-	65 cfs at elevation 91.9 feet.

5. DAM

Type	-	Earthfill.
Hydraulic Height	-	45.9 feet crest to toe.
Crest Elevation	-	91.9 feet.
Crest Width	-	950 feet.
Upstream Slope	-	Variable 1V on 3H to 1V on 3.5H.
Downstream Slope	-	1V on 2.5H.

CHAPTER 1

BACKGROUND

1.1 INTRODUCTION.

1.1.1 Authority and Scope.

This report summarizes the Phase I inspection and evaluation of Vaux #1 and Vaux #2 Dams owned by the Lone Tree Ranch Corporation of Sidney, Montana. The dams are also often referred to as the Anderson Dams.

The National Dam Inspection Act, Public Law 92-367 dated 8 August 1972, authorized the Secretary of the Army, through the Corps of Engineers, to conduct safety inspections of non-federal dams throughout the United States. Pursuant to that authority, the Chief of Engineers issued "Recommended Guidelines for Safety Inspection of Dams" in Appendix D, Volume 1 of the U.S. Army Corps of Engineers' Report to the United States Congress on "National Program of Inspection of Dams" in May 1975 (reference 8).

The recommended guidelines were prepared with the help of engineers and scientists highly experienced in dam safety from many federal and state agencies, professional engineering organizations and private engineering consulting firm. Consequently, the evaluation criteria presented in the guidelines represent the comprehensive consensus of the engineering community.

When necessary the guidelines recommend a two-phased study procedure for investigating and evaluating existing dam conditions so deficiencies and hazardous conditions can be readily identified and corrected. The Phase I study is:

(1) a limited investigation to assess the general safety condition of the dam.

(2) based upon an evaluation of the available data and a visual inspection.

(3) performed to determine if any needed emergency measures and/or if additional studies, investigations and analyses are necessary or warranted.

(4) not intended to include extensive explorations, analyses or to provide detailed alternative correction recommendations.

The Phase II investigation includes all additional studies necessary to evaluate the safety of the dam. Included in Phase II, as required, should be additional visual inspections, measurements, foundation exploration and testing, material testing, hydraulics and hydrologic analyses and structural stability analyses.

The authority for the Corps of Engineers to participate in the inspection of non-federally owned dams is limited to Phase I investigations with the exception of situations of extreme emergency. In these cases the Corps may proceed with Phase II studies but only to the extent needed to answer serious questions relating to dam safety that cannot be answered otherwise. The two phases of investigation outlined above are itemed only to evaluate project safety and do not encompass in scope the engineering required to perform design or corrective modification work. Recommendations contained in this report may be for either phase II safety analysis or detailed design study for corrective work. The responsibility for implementation of these recommendations rests with the dam owner and the State of Montana. It should be noted that nothing contained in the National Dam Inspection Act, and no action or failure to act under this act shall be construed (1) to create liability in the United States or its officers or employees for recovery of damage caused by such action or failure to act or (2) to relieve an owner or operator of a dam of the legal duties, obligations, or liabilities incident to the ownership or operation of the dam.

1.1.2 Purpose.

The purpose of the inspection and evaluation of Vaux #1 and Vaux #2 Dams is to determine if conditions exist which constitute a danger to human life or property so that timely corrective measures can be taken by non-Federal interests.

1.1.3 Inspection.

The findings and recommendations presented in this report are based upon an evaluation of the project performed by Morrison-Maierle, Inc., Consulting Engineers of Helena, Montana. The evaluation was based on observations from an on-site inspection conducted on September 27, 1978, a review of available drawings and bid documents for reconstruction of the dams, interviews with the owners, review of historical photographs in the possession of Mr. S.A. Anderson, and review of published geologic and hydrologic data for the region. The inspection and evaluation were performed in accordance with the "Recommended Guidelines for Safety Inspections of Dams".

The field inspection team consisted of the following personnel:

Rodger C. Foster, P.E. - Civil Engineer, Team Leader, Morrison-Maierle, Inc.;

Mike Kaczmarek - Geologist, Morrison-Maierle, Inc.;

David Barnett - Structural Engineer, Morrison-Maierle, Inc.;

Glen McDonald - Dam Safety Engineer, Montana Department of Natural Resources and Conservation.

Also present during the inspection representing the Lone Tree Ranch Corporation were:

Mr. S.A. Anderson of Sidney, MT

Dr. Hugh Anderson, M.D. of Great Falls, MT

Mrs. Ruth A. Witt of Sidney, MT

Mr. Bryce Witt of Sidney, MT - Operator

Additional Morrison-Maierle personnel involved in the evaluation were:

Mr. Harold Eagle, P.E. - Sr. Vice President and Chief Engineer, Soils and Report Review

Mr. Ken Salo - Hydrology and Hydraulics

1.2 DESCRIPTION OF PROJECT.

1.2.1 General.

Vaux #1 and Vaux #2 Dams are owned and operated by the Lone Tree Ranch Corporation as an irrigation water supply project. These earthfill dams are located about three miles upstream of the Town of Sidney, Richland County, Montana on Lone Tree Creek in Township 23 North, Range 58 East, Sections 25 and 26, respectively. Their total drainage is approximately 83 square miles. The dams are operated as one project with Vaux #2 located about one-half mile upstream of Vaux #1 Dam. Photographs 1 and 2 and Plates 1, 2, 3 and 5 provide a good description of the location and terrain of the project area.

Vaux #2, the upper dam, is used as primary storage and regulates flow only to the lower reservoir. A spillway at Vaux #2 provides the only flood passing capabilities for the project. Vaux #2 Dam is 46-feet high and impounds approximately 1200 acre-feet at the dam crest elevation.

Vaux #1, the lower dam, provides additional project storage and serves as the only regulation for the irrigation releases. It has no functional spillway. Vaux #1 Dam is 24-feet high and impounds approximately 685 acre-feet at the dam crest elevation.

Based on a visual reconnaissance and engineering judgement, failure of the Vaux #1 and #2 dams would cause severe downstream flooding, property destruction and the potential for loss of life as the flood surge traveled down the valley into the town of Sidney.

1.2.2 Regional Geology.

The Vaux dams are located within the southernmost limit of continental glaciation where a lobe of the continental ice sheet pushed southward into the ancestral Yellowstone River Valley. Surficial deposits include preglacial, glacial and postglacial materials.

In an overview, the terrain of the Vaux dams sites consists of an upland of ruggedly eroded bedrock strata protruding through a plain of glacial outwash gravels that overlie glacial till. The glacial till resting on the buried preglacial landscape is usually in contact with bedrock. The glacial till is separated from the bedrock locally by buried preglacial gravel.

The bedrock underlying the dam and reservoir sites, as well as most of the contributing watershed, consists of continental basin deposits of the Tertiary Fort Union Formation. The Fort Union Formation consists of fine-to very fine-grained silty sandstones, soft siltstones, mudstones, claystones, bentonite, lignite and red and gray scoria and clinker beds.

The Fort Union beds are lenticularly bedded and individual lenses range from several hundred to several thousand yards in extent. The continuity of individual lenses may not be relied upon for stratigraphic correlation or structural interpretation across covered intervals. Examination of Fort Union beds exposed in the Lone Tree Creek valley walls downstream from the dam and reservoir sites revealed no evidence of localized faults or structural deformation. Geologic structure at the two dam sites is limited to regional warping and deformation.

Preglacial materials are terrace deposits consisting of cohesionless sand and gravel. The terrace gravels are discontinuous beneath the glacial deposits and represent remnants of preglacial stream channels and associated terraces present on the ancestral preglacial landscape carved into the surface of the Fort Union Formation. The preglacial terrace gravels are covered by an essentially continuous overburden of glacial till that rests on Fort Union bedrock where preglacial gravels are absent. The glacial till consists of a matrix of silty clay containing sand through gravel size detrital material. The glacial till is densely compact and preconsolidated and remolds into a highly plastic soil with a moderately high shrink-swell ratio.

Related glacial deposits are glacial outwash consisting of stratified sand and gravel deposited as terraces on top of the glacial till and in channels cut into the glacial till. The outwash sand and gravel was deposited by melt water flowing away from the melting continental ice sheet. Postglacial deposits consist of unconsolidated alluvial sand and gravel present in the bottom of the modern stream and river valleys. The alluvium is derived largely from the upland outwash gravel deposits but also contains lenses of silts and clays derived from the glacial till and the Fort Union bedrock. Silty and clayey lake sediments may be present beneath the alluvium and some of the low level outwash terraces as a result of sedimentation in ice marginal lakes.

The modern Lone Tree and Brorson Creeks generally follow buried preglacial channels, however, the thick glacial till in the NE¼, Section 26, Township 23 North, Range 58 East, probably represents a buried preglacial channel filled by till. The glacial outwash gravels are present at several terrace levels including one inside the modern Lone Tree channel. An outwash terrace at elevation 2150 is distinctly marked by a prominant esker of outwash gravel north of the Vaux reservoir sites that is indicative of the recessional origin of the outwash gravels.

1.2.3 Seismicity.

The Vaux Dam and reservoir sites are located in seismic probability Zone 1. The seismic probabilities used herein divide the United States into four seismic risk zones based on the record of the severity of ground shaking and reasonable expectancy of earthquake damage. Seismic probability or risk in Zone 1 indicates that earthquakes capable of minor damage may occur. Zone 1 corresponds to a potential intensity of V and VI on the Modified Mercalli intensity scale of 1931. Significant earthquake shocks have not occurred in the vicinity of the Vaux Dams within the period of recorded earthquake history for the area that dates back to about 1900.

1.2.4 Site Geology.

1.2.4.1 Vaux Dam #1. The right abutment of the Vaux Dam #1 consists of glacial till resting against Fort Union bedrock (Plate 11). The glacial till is estimated to be 25 to 50 feet thick and extends an unknown depth below the abutment. The Fort Union formation exposed behind the glacial till consists of a soft, friable, very-fine to fine-grained sandstone. The top of a lignite bed was exposed 2.0 feet above the water surface in the reservoir at the time of the inspection. The base of the lignite bed extended below the water surface at the time of the inspection.

Wave action has undermined the glacial till in the reservoir area adjacent to the right abutment. The wave undermining is evidently the triggering mechanism of a small active rotational shear slump in the glacial till. Pore pressure from seepage moving through the lignite bed in the Fort Union bedrock may also aggrevate the slump action. The slump is approximately 100 feet long and apparently recurrent slump movements have resulted in vertical displacement of about 6 to 8 feet. The slump poses no threat to the structural stability of the embankment or to overall dam safety and operation.

The left abutment of Vaux Dam #1 consists of a ridge of glacial till that separates Lone Tree and Brorson Creeks slightly upstream from their confluence (Plate 11). The glacial till is estimated to be thick at this location and is stable. The dense, compact till is preconsolidated and appears to provide a satisfactory dam abutment.

The Lone Tree Creek Valley floor at the dam is covered with alluvium consisting of sand and gravel of unknown thickness. The sand and gravel alluvium is the foundation under the main fill of the 1936 dam enbankment and under most of the 1960 embankment. The 1954 engineering plans (Plates 4 and 7) indicate a 10-foot deep core trench under the new fill added to the 1936 embankment that remained after the 1951 washout. It is not known what type of material the core trench is founded in. Field inspection of the dam reveals no evidence of settling cracks, mud heaving or misalignment that might indicate foundation instability.

1.2.4.2 Vaux Dam #2. Plate 11 schematically portrays the general geologic conditions at the abutments of the Vaux Dam #2. The right abutment consists of fine-to very fine-grained friable sandstone and dense claystone and mudstone units of the Fort Union Formation. It is not known if the dam embankment is fully keyed through a small terrace of glacial outwash on the right abutment. The left abutment consists of an unknown thickness of glacial till resting on the Fort Union bedrock. Glacial till exposed in the left shore of the reservoir about 300 yards upstream from the left abutment ranges between 10 and 20 feet thick. Glacial till exposed in the left valley wall about 500 yards downstream from the left abutment is the same range of thickness. Elevations established by leveling along the top surface of the glacial till adjacent to the left abutment, indicate that the upper 5 feet of the Vaux #2 Dam embankment abuts glacial outwash gravels as shown on Plate 11.

The left abutment contiguous to the upstream face of the dam is the location of the spillway of the 1947 dam. The 1947 spillway has been backfilled with silty sandy gravel apparently borrowed from the glacial outwash gravels. The left abutment contiguous to the downstream face of the dam exhibits a small rotational shear slump. The shear failure is in glacial till and overlying glacial outwash gravels have rolled down with the slump block creating a debris flow that includes chaotic blocks of till, loose sand and gravel and detachments of sod. Photographs of the abutment taken by Mr. Sig Anderson in 1951 show a vertical scarp as a result of flood erosion of the abutment. Over-steeping appears to be the primary factor causing the slump location. The slump has no potential to affect the embankment or spillway functions of Vaux Dam #2 and does not present a structural stability or safety hazard.

The Lone Tree Creek valley at the downstream toe of the Vaux #2 Dam is filled with sand and gravel alluvium that is traversed by channel scars. The thickness of the alluvium is unknown. Indications are that the 1944 and 1947 embankments were founded directly on the alluvium with very little site preparation. The 1960 repair section constructed in the washed out portion of the dam (plates 4 and 6) includes a cutoff trench excavated 10 feet into the alluvium.

Mr. Sig Anderson describes the material at the bottom of the cutoff trench as a "heavy blue clay" and provided a sample of remolded clay collected from the trench during the 1960 reconstruction. The clay ball sample consists of a heavy plastic clay, dark gray when moist, that contained no detrital silt or sand. The clay exhibited the somewhat fibrous texture of ashy clay such as bentonite or partially devitrified volcanic ash. Clay of this type was observed in descrete strata of the Fort Union Formation during the field inspection; however, the sample could also be remolded glacial lake sediment or even a clay plug (unlikely) in the alluvium. The cutoff under the 1960 embankment may be founded in impermeable foundation strata; however, the nature of the impermeable strata is unknown except as described by Mr. Anderson.

Field inspection of the alluvium downstream from the dam revealed no sign of mud heaving, toe bulges, or other evidences of inadequate shear strength in the foundation materials. Foundation seepage was noted and is discussed in section 2.4.2.2 of this report.

1.2.5 Design and Construction History.

Vaux #1 Dam was first constructed in 1936 and was approximately 20 feet high. An open channel spillway which discharged to Brorson Creek was constructed adjacent to the Vaux #1 Dam near the north abutment. This spillway was later abandoned when a road was constructed across the entrance.

Vaux #2 was constructed in 1944. In 1947 it was raised to a height of 40 feet and a spillway was constructed at Vaux #2. This spillway, noted as "old spillway" on Plate 5, was formed by cutting an open channel through the dividing ridge between Lone Tree Creek and Brorson Creek.

On March 26, 1951, both Vaux #2 and Vaux #1 Dams failed due to overtopping during a high runoff. A description of the failures is presented in the following section. Lillis Engineering of Billings, Montana, was retained to design repairs to the dams and prepared a brief document entitled "Plans and Specifications for Repairing Florence V. Anderson Dams", dated January 1954. The plans from that document are reproduced on Plates 4 through 7. It should be noted that the Vaux Dams were originally referred to as the Anderson dams and on the plans for repair (plates 4-7) are referred to as S.A. Anderson Dams. The reconstructed dams were renamed as a memorial to the late August Vaux.

Reconstruction of the dams was completed in 1960 by Buller Construction Company. The firm of Burdict & Webster of Williston, N.D., supervised the construction. The reconstruction involved replacement of the outlet works at both dams, rebuilding of breached sections of the dams, and construction of a new spillway at Vaux #2 Dam. The dams now exist as reconstructed in 1960.

The only document available concerning the design of the Vaux Dams at any stage are the plans and specifications prepared by Lillis Engineering. Mr. S.A. Anderson has a personal photo file showing the dams at various periods and has photo coverage of the breached embankments and reconstruction work.

1.2.5.1 Dam Failure 1951. On March 21, 1951 the Vaux Dams failed causing extensive flooding downstream and in the southern portions of the Town of Sidney. High flows occurred on Lone Tree Creek prior to the breakup of the ice on the upper reservoir. Ice jammed at a wooden bridge which the county had constructed across the spillway channel collapsing the bridge and causing a great reduction in the capacity of the spillway. Vaux #2 Dam was overtopped and after "several hours" a 200-foot long section at the north abutment was washed out. The failure of the upper dam caused Vaux #1 to be overtopped and a section about 400 feet long at the south abutment was washed out. Inflow to Vaux #2 at the time of the failure was estimated to be no more than 2000 cfs. Although the failure caused significant damage in the Town of Sidney, there was no loss of life.

1.2.6 Physiography, Climatology and Hydrology.

Vaux #2 Dam is located on Lone Tree Creek in the Yellowstone River Basin at longitude 104° 14' 08" and latitude 47° 43' 48". At normal operating pool (dam datum elevation 81) the pool elevation is approximately 2090 MSL as estimated from a USGS Sidney, Montana Quadrangle Map. Vaux #1 is located at Longitude 104° 13' 30" and Latitude 47° 43' 40" and is operated at a normal pool elevation of approximately 2063 MSL. Maximum elevation in the drainage is approximately 2640 feet. The 83 square mile drainage is approximately 16 miles long and 8 miles wide with a longest stream length of 24.3 miles. Ground cover in the drainage is primarily range grasses with over 90 percent of the soils classed as having moderate to slow infiltration rates (SCS Letter 3/5/71). The drainage receives an average of 14 inches of precipitation annually in the form of rain and snow.

CHAPTER 2

INSPECTION AND RECORDS ELEVATION

2.1 SPILLWAY DESIGN FLOOD.

The Vaux Dams project consists of two separate dams located approximately one-half mile apart on Lone Tree Creek. The dams are operated as one project. The lower dam, Vaux #1, is operated as a regulating reservoir and has virtually no contributing drainage area. All inflow to the lower reservoir must pass through Vaux #2 Dam outlet works. The upstream dam is the larger of the two dams and serves as the primary storage reservoir with the only spillway capabilities for the project. The spillway design flood (SDF) for the project is therefore based on the classification of Vaux #2 Dam since the hydraulic and hydrologic features of the upper dam will determine the safety and hazard of the overall project. Inspection criteria recommend the spillway design flood to be a full probable maximum flood (PMF).

2.2 HYDRAULICS AND STRUCTURES.

2.2.1 Vaux #1 Dam

2.2.1.1 Vaux #1 Outlet Works. The outlet from Vaux #1 is a 24-inch diameter, gate-controlled, concrete conduit approximately 150 feet long. The conduit extends through the embankment near the south abutment. It is controlled by a slide gate located in a gate shaft which rises vertically to the crest at the center of the dam (Plate 7). Photographs of the reconstruction show that concrete collars were placed around the conduit as shown on Plate 7.

The slide gate is operated by a gate stem connecting to a control wheel mounted on the concrete cover to the gate shaft. During the inspection, the owners related that they had experienced problems with the gate binding and preventing closure. When the gate control wheel was operated, rather than closing the gate it would cause the concrete cover to be lifted out of place. According to the owners corrective repairs have been made to the gate. The gate shaft is a wet well and could not be inspected below approximately seven feet from the cover due to the level of the water. There was no evidence of concrete deformation on the visible shaft walls.

The outlet conduit was inspected at the exit and was in good condition. No concrete deterioration or displacement of joints was observed from the exit, however, the entire conduit was not inspected. The conduit discharges to a flow diversion structure as is shown in photo 36. There is no indication of erosion around the conduit exit. At maximum pool the capacity of the outlet is approximately 50 cfs.

2.2.1.2 Vaux #1 Spillway. An open channel spillway which was constructed with the original dam still exists near the north abutment where it was cut through the natural divide to Brorson Creek (Photo 33). After the construction of the upper dam the spillway at Vaux #1 was abandoned and two earth embankment roads were constructed across the channel. The minimum elevation of the roads is one foot higher than the top of the dam which prevents the spillway from functioning. The spillway channel has a bed width of approximately 100 feet and a maximum bed elevation 4 feet below the crest of the dam. The channel extends 235 feet on a 0.5% slope to a concrete sill and then drops to Brorson Creek. Normal operating pool is seven feet below the crest of the dam, however, high water marks indicate the pool has been operated with five feet of freeboard.

Because Vaux #1 has no emergency spilling capabilities a situation exists which could cause the dam to fail even under normal operating conditions. If, for any reason, flow were discharged from Vaux #2 outlet works for a long enough period without a corresponding discharge from Vaux #1, the reservoir would fill and overtop the embankment creating a potentially hazardous condition.

2.2.1.3 Vaux #1 Freeboard. The Vaux #1 reservoir is regulated by releases from Vaux #2 and has no significant inflow from natural runoff. The dam overtops during the PMF due to the failure of Vaux #2 and therefore there is no freeboard with PMF conditions. During normal operations, the reservoir is maintained with approximately 7 feet between the pool and embankment crest. This elevation difference is adequate to prevent overtopping from wind-generated waves during normal pool conditions.

2.2.2 Vaux #2 Dam.

2.2.2.1 Vaux #2 Outlet Works. The outlet for Vaux #2 was constructed in 1960 during the reconstruction of the breached section of the dam at the north abutment and consists of a 270-foot long 24-inch diameter concrete conduit with a slide gate control at the center of the dam (Plate 6). Mr. Sig Anderson's personal photographs of the construction show cutoff collars were formed around the conduit as indicated on Plate 6. Inspection of the conduit at the exit showed it to be in good condition with no apparent displacement or deterioration, however, the conduit itself was not inspected. There is a concrete apron extending 21 feet beyond the exit which is cracked, broken, and displaced (Photo 14). The condition of the apron has allowed the fill to erode from beneath the apron and around the collar at the outlet. Measurements taken during the inspection show the flow line at the exit of the conduit to be at elevation 51.2 feet rather than at elevation 46 feet as is indicated on the preconstruction drawing on Plate 6. Elevation 46 feet is actually the elevation of the streambed below the outlet apron. This discrepancy is not signficant and could easily be reflective of modifications made during construction.

The slide gate is located in a 48-inch diameter manhole gate shaft which rises to the crest of the dam. The gate control consists of a gate stem connected to a wheel mounted on the concrete cover to the gate shaft (Photo 12). According to the owners, the slide gate binds during closure but has not prevented operation of the gate. The gate was operated during the inspection and appeared in good operating condition. At maximum pool the outlet capacity is approximately 65 cfs.

2.2.2.2 Vaux #2 Spillway. The project has one spillway located on the Vaux #2 reservoir. The spillway is an open channel cut through the natural divide separating Lone Tree Creek and Brorson Creek. The alignment of the spillway, as shown in Photo 1 and Plate 5, diverts water from the upper reservoir to Brorson Creek bypassing the lower reservoir. The spillway is an unlined flat bottomed trapezoidal channel with a bed width of approximately 45 feet and 2 to 1 side slopes. The channel slope is essentially flat for the first 600 feet and then continues on a .3% slope for the next 700 feet. Thirteen hundred feet downstream from the spillway entrance the channel is extremely eroded (Photos 17 & 18). Average channel depth in the noneroded section is approximately 14 feet. The entrance to the spillway channel (Photos 15 & 16) is located approximately 400 feet upstream of the north abutment in a small natural cove approximately 115 feet across. There is no structural control in the spillway channel and the highest channel bed elevation is 11 feet below the crest of Vaux #2. There is no structural means of energy dissipation.

A hydraulic analysis of the spillway was made using "HEC-2 Water Surface Profiles" (reference 7) to determine the capacity of the channel. For the analysis, cross sections of the spillway channel were determined at 300 foot intervals from the eroded section at station 13+38 to the reservoir. A profile of the channel is shown on Plate 9. Critical depth was assumed as a starting water surface elevation where erosion has caused a break in bed slope. Using Manning's roughness coefficients of .030 to .035, water surface profiles were computed for a range of flows to develop a stage-discharge relationship for the spillway, (Plate 9). These computations show, when the reservoir level is at the top of the dam, the flow in the spillway would be approximately 4350 cfs.

Accelerated erosion beyond station 13+38 of the spillway is evidenced as a 37 foot deep gully cut in the channel bed. The gully cuts to the approximate elevation of Brorson Creek to which the spillway discharges. The gully is progressing upstream by headward erosion and is, according to the owner, the result of three or four "major spills", since 1960. The spillway and associated gully erosion are in dense glacial till which is relatively resistant to erosion. However, flow velocity at the critical section in the spillway ranges between 8 and 14 feet per second for discharges of 1000 to 4000 cfs respectively. At these velocities, continued erosion of the channel would be expected, however, the corresponding discharges would be considered "major spills" occuring rather infrequently. If the erosion progresses up

the spillway to approximately station 4+00 at the location of the old spillway (Plate 5), it is possible the stability of the dam's north abutment would be affected.

2.2.2.3 Vaux #2 Freeboard. When Vaux #2 was rebuilt in 1960 it was raised to provide an increased margin of safety against a recurrence of the failure experienced in 1951. The dam overtops during the PMF, therefore there is no freeboard with PMF conditions. During normal operations, the reservoir is maintained below the crest of the spillway thereby providing 11 feet between the pool and the embankment crest. This elevation difference is adequate to prevent overtopping from wind-generated waves during normal pool conditions.

2.3 HYDROLOGY.

2.3.1 Reservoir Stage and Discharge.

The elevation-storage curves presented on Plate 8 were developed using cross-sections approximated from a USGS topographic quadrangle map. The only discharge considered for the project was that of the spillway at Vaux #2 Dam. Discharges from the outlet works at both dams were considered inconsequential to the analysis. The only major inflow to the Vaux #1 Reservoir would be the result of a failure of the Vaux #2 Dam.

2.3.2 Preliminary Probable Maximum Flood (PMF).

The spillway design flood for the Vaux Dams project is a full PMF. As part of the determination of the PMF, probable maximum precipitation (PMP) was developed following procedures outlined in "Hydrometeorological Report No. 51" (reference 11) which applies to areas of the United States east of the 105th meridian. The critical storm was selected as being a June storm of 72 hours duration. Arranging the storm to provide a critical time sequence of one hour intervals yielded a PMP of 23.8 inches.

To determine the probable maximum flood, the PMP was applied to unfrozen but saturated ground using a minimum constant loss rate for class B hydrologic soils (moderate infiltration) of 0.15 inches per hour. A unit hydrograph was developed for the watershed using procedures presented in the U.S. Bureau of Reclamation publication "Design of Small Dams" (reference 12). The Corps of Engineers' computer program HEC-1 Flood Hydrograph Package (reference 6) was used to combine the PMP data with the unit hydrograph to develop a PMF reservoir inflow hydrograph. The resulting PMF had a peak flow of 92,800 cfs with a run-off volume of approximately 77,500 A-F.

2.3.3 Routing of the Preliminary Probable Maximum Flood.

2.3.3.1 Vaux #2 Dam. Routing of the probable maximum flood through the Vaux #2 Reservoir was performed using the computer program HEC-1. The routing analysis assumed an initial pool elevation at the spillway crest elevation. The capacity of the outlet works was

not considered in the routing because of its relatively insignificant capacity and because it is possible the operator would not be available to open the gate during a peak event.

The routing analysis of the PMF indicates the dam would be overtopped early in the flood. Flood routings were also performed for much smaller events and they indicate the project is only capable of safely handling a flood equivalent to approximately 5 percent of the PMF. At higher flows the reservoir has little routing effect.

2.3.3.2 Vaux #1 Dam. Vaux #1 Dam has no functional spillway and its routing capabilities depend entirely on the outlet works and excess storage capacity. The only major source of inflow to Vaux #1 during a PMF is from overtopping and failure of Vaux #2 Dam. Because Vaux #1 has a much smaller storage capacity than Vaux #2 and has no functional spillway, Vaux #1 will also be overtopped during a PMF.

2.4 GEOTECHNICAL EVALUATION.

2.4.1 Dam Embankments.

2.4.1.1 Vaux #1 Dam Embankment. Vaux #1 Dam was constructed in 1936. The height of the 1936 embankment was reportedly 20 feet. Little is known regarding the nature of the fill materials or the methods of emplacement and compaction used in constructing the 1936 embankment. The 1936 embankment remained in place until March 26, 1951 when overtopping and failure of the Vaux #1 Dam resulted in erosion and washing out of about 400 feet of the approximately 800 feet long 1936 embankment. In 1960, Vaux #1 was reconstructed.

Engineering plans and specifications prepared in 1954 by Lillis Engineering for the reconstruction contract show that two design sections were used for the reconstructed embankment (Plate 4). One design section incorporates the remaining 1936 embankment into the new fill by means of emplacement of new impervious fill on the downstream face and construction of a pervious protective gravel blanket over the new fill.

The second design section used on the 1960 embankment at Vaux #1 Dam is a completely new design used to replace the washed out section. The design consists of an impermeable core protected by permeable sand and gravel outer shells. A 3-foot thick upstream clay blanket is also included under the embankment in this design. Both design sections for the 1960 repairs include 6" diameter toe drains in the permeable downstream shells.

Field measurements and visual inspection of the Vaux #1 Dam embankments reveals embankment dimensions conforming closely to the 1954 Lillis Engineering drawings (Plates 4 & 7). The embankment is approximately 800 feet long from abutment to abutment. The crest breadth is 20 feet and average embankment height is 24 feet measured from the invert of the outlet conduit. The downstream backslope is

1V on 2.5H on the 1960 embankment and 1V on 2H on the rehabilitated 1936 embankment. The difference in the two backslopes was noted during the field inspection and caused some confusion until the difference was detected between the 1954 and the 1936 embankment designs. The upstream backslope was underwater and could not be measured.

The source of fill for the impermeable zones constructed in 1960 was evidently glacial till from the right abutment and Fort Union claystone and mudstone. The source of impermeable material for the 1936 embankment was probably glacial till from the currently abandoned emergency spillway. It appears likely that significant amounts of outwash sand and gravel were mixed with the clayey borrow to make up the impermeable zone fill. Field examination of the glacial till and Fort Union Formation materials indicates that both are clayey soils and are well suited for use as an impermeable earthfill embankment core material. Detailed photographs of the 1960 construction activities, taken by Mr. Sig Anderson, show the fill being placed in successive lifts, sprinkled, bladed, and compacted with a sheepsfoot roller. The source of the outer protective shells of sand and gravel is glacial outwash.

The gravel shell surfaces of the dam are bearing up well under 18 years of performance on both faces of the dam and show no sign of instability or damage except for a wave cut bench on the upstream face. Wave action during the past 18 years has cut a bench in the upstream embankment face of Vaux #1 that has a 4 to 5 foot high vertical scarp (Photo 7). With the absence of riprap protection on the upstream dam face, wave erosion could be expected to continue. Also surface drainage from the road at the right abutment is directed down the upstream face of the dam at the point of contact between the dam and natural ground (photos 26 and 27). This area is heavily grassed and no erosional damage to the embankment has resulted.

2.4.1.2 Vaux #2 Dam Embankment. Vaux #2 is an earthfill dam approximately 46 feet high as measured from the bed of the outlet channel, and is approximately 950 feet long. The crest breadth is 20 feet. Field measurements taken during the field inspection verify the basic dimensions of the dam as they are shown on Plate 4.

The key elevations determined in the field and summarized in Table 2 are consistent with the datum shown on preconstruction drawings in Plates 4 and 6 using the top of the concrete cover to the gate shaft as a bench mark reference elevation. Elevations determined in the field show that the crest of the dam is actually one foot higher than the elevation of 91 feet shown on the drawings. This discrepancy is not significant and could easily be reflective of modifications made during construction.

"Plans and Specifications for repairing the Florence V. Anderson Dams" prepared by Lillis Engineering in January, 1954, indicate that the initial Vaux #2 Dam embankment was constructed in 1944. In 1947, the embankment was raised to a height of about 40 feet and a spillway was excavated into the Brorson Creek Valley. Information regarding the design parameters, foundation investigations or conditions, and methods of construction was not recorded. The Lillis Engineering plans and specifications state:

> "In connection with the construction of the original dams, the customary borings were made to determine the foundation conditions; and these conditions were considered satisfactory and were found to be so by reason of the dams standing for from 6 to 15 years without foundation trouble or serious seepage."

Engineering drawings prepared by Lillis Engineering in 1954 (Plate 4) show that the 1947 embankment (presumably rolled earth) remaining after the flood damage in 1951 was incorporated into the 1960 embankment by adding protective gravel shells to the upstream and downstream faces to bring them to the present backslopes. The plans also call for a 6" diameter toe drain in the downstream shell. No evidence of the toe drain could be detected during the field inspection. No cutoff or upstream blanket is specified for the rehabilitation of the 1947 embankment in the plans presented for the 1960 reconstruction.

The same drawings (Plate 4 & 6) show a design section to replace the washed out portion of the dam to consist of an impermeable core and cutoff protected by permeable shells on the upstream and downstream faces including a 3-foot thick blanket under the upstream shell. Field inspection shows the "permeable" outer shells to consist of sandy gravel derived from the local glacial outwash deposits. A portion of the impermeable core material is exposed in a small washout behind the downstream collar on the main outlet tube. The material consists of clay derived mainly from glacial till with an estimated relative compaction of 90 to 100 precent (based on field penetration estimated from geologic pick blows in the moist material).

Photographs of the 1960 construction activity, provided by Mr. Sig Anderson, show the impermeable fill material being placed in lifts by scrapers, sprinkled, bladed, and compacted with a sheepsfoot roller. The photographs also show excavation of the cutoff trench and show that the new 1960 embankment fill is keyed into the older 1947 embankment.

Evidence of differential settlement, slope failure, or misalignment was not observed in the embankment. The protective outer shells of gravel are undamaged by animal traffic or burrows and show no signs of erosion with the exception of a wave cut bench in the upstream face. The wave cut bench scarp is 5 to 6 feet high after 18 years of wave

erosion and does not penetrate to the inner core material of the dam. With the absence of riprap protection, wave erosion could be expected to continue.

2.4.2 Foundation Conditions.
2.4.2.1 Vaux #1 Dam Foundation Conditions and Seepage. Careful inspection of the downstream face of the Vaux #1 Dam revealed free-flowing seepage at the toe of the embankment in about a 50-foot wide area 250 feet from the right abutment. In addition, a distinct zone of phreatic seepage development is delineated by the presence of sedge grass growth and wetland vegetation development that covers the downstream face of the right one-half of the dam to a height of about 10 vertical feet above the toe of the fill (Photos 24 and 25).

The wetting front is confined to that part of the embankment containing remains of the 1936 dam and is interpreted to represent the phreatic seepage line through the embankment at maximum operating storage pool in this portion of the dam. In comparison to the vegetation zone, the free flowing seepage extended only to 4 or 5 vertical feet above the toe of the fill at the time of the inspection. Six inch diameter drains shown on repair drawings were not located during the inspection. There is no evidence of piping or slope failure in the embankment as a result of the seepage. However, the location of the phreatic line and the resultant pore pressures are a critical consideration in the embankment stability and the observed conditions indicate that the dam does not meet minimum factors of safety for the downstream slope. Also the observed high phreatic surface on the downstream slope indicates that the gravel shell material is actually not very pervious and the 6-inch drain shown on the repair plans is not effective.

Seepage was not observed through the lignite seam in the right abutment due to the relatively impervious cover of glacial till, however, unusually dense growth of brush on the glacial till on the downstream side of the right abutment suggests a source of anomalous soil moisture content in the glacial till at that location. There are no piezometers or other instrumentation in the dam or abutments.

2.4.2.2 Vaux #2 Dam Foundation Conditions and Seepage. Field inspection of Vaux #2 Dam detected seepage in two locations. Seepage was issuing as clear, free-flowing water from the contact of the embankment with the north (left) abutment beginning at an elevation approximately 32 feet below the average crest elevation of the dam. The seep showed no evidence of piping or erosion and was apparently not related to the nearby slump in the glacial till.

The rate of discharge at the time of the inspection was a steady trickle of less than 0.5 gallon per minute. The seepage zone was well vegetated by broadleaf grass but phreatophytes and typical marsh or wetland grasses and plants were absent with the

exception of "snakegrass". The source of water for the seep is unknown. It cannot be discerned whether the seepage flow path was along the embankment-abutment interface or through a permeable sand lense in the abutment. The fact that the seepage surfaces, rather than being drained away through the subsurface of permeable gravel shell on the face of the dam indicates that the gravel shell is not very permeable in this area.

The second area of seepage detected by the field inspection is in the foundation near the south (right) abutment (Plate 11) and shows clearly in photographs 8, 9, and 10 as an area of lush grass. Seepage discharge from this area was estimated at 20 to 30 gallons per minute in a small channel plus additional free-flowing seepage over a broad area at the time of the field inspection. Examination of the geologic materials in shovel holes and old backslopes in a continuous borrow area reveals the alluvium (or outwash terrace) in this location to contain a high percentage of fine-grained sand lenses as much as 4 to 5 feet thick.

The seepage area is in the foundation of the rehabilitated 1944 and 1947 embankment where a cutoff was probably not used. The seepage begins at the toe of the embankment, however, the most vigorous discharge takes place 20-25 feet downstream from the embankment and seepage extends as far as 70 feet downstream from the dam. Moreover, the seepage seems to be confined to a low terrace approximately 15 feet higher than the mean surface of the alluvial floor and 23 feet below the average dam crest elevation. The terrace is either a high, remnant alluvial terrace or a low-level glacial outwash terrace. Seepage is not evident in the main alluvial valley floor sediment and may be restricted to a permeable lense in the low outwash terrace.

The seepage area is well-vegetated with lush grass. Shovel holes penetrate 1 to 2 feet of sandy organic soil before encountering 4 to 8 inch cobbles. Flow from the seepage area is clear and no evidence of piping or accelerated erosion is present. The direction of discharge from the seepage area is from the top of the low bench towards the main valley floor and so is parallel to the face of the dam. Thus, the small amount of integrated channel development that is present does not tend to erode toward the toe of the dam.

At the time of the inspection the differential head between the reservoir storage level and the headwater in the seepage area was approximately 11.5 feet. Under this differential pressure, a man jumping on the surface of the seepage area was able to cause the ground to heave and quiver in waves radiating for 8-10 feet in all directions. This is demonstrative of the hydrostatic uplift pressure in the foundation at this location. Piping and erosion of subsurface materials have apparently been controled by the dense mat of grassroots stabilizing the seepage front soils and by the presence of coarse-grained sediments, particularly the cobbles, in the seepage path. The presence of a number of loose cobbles at the toe of the low terrace

below the seepage zone suggests that some surface piping or erosion may have taken place prior to being stopped by the larger particles in the terrace and the subsequent development of a stabilizing grass mat. There are no piezometers or other instrumentation in the dam or abutments. No evidence of the 6-inch drains shown on the repair plans was observed.

2.4.3 Embankment Stability.

2.4.3.1 Vaux #1 Dam Embankment Stability. Data for shear strength, pore pressure, permeability, and unit weight apparently do not exist for the Vaux #1 Dam embankment and records of a design stability analysis could not be found. Two conditions were observed which indicate the need for additional field investigation and testing:

(1) The presence of free-flowing seepage from the toe of the rehabilitated 1936 embankment, and

(2) The presence of plant growth evidently associated with phreatic seepage or a wetting front to an elevation 10 vertical feet above the downstream toe elevation in the rehabilitated 1936 embankment.

The observed seepage conditions indicate the presence of high pore pressures which adversely affect the embankment stability at normal maximum operating pool or at higher levels of storage. Experience with other similar structures indicates that the safety factor of downstream slope does not meet minimum guideline requirements. Appropriate field investigations including piezometer installation and monitoring, combined with an embankment stability analysis are required to evaluate the nature and effect of seepage pressure in and under the Vaux #1 Dam embankment.

"Recommended Guidelines For Safety Inspection of Dams" do not require embankment stability analysis under seismic loading conditions if static stability conditions provide satisfactory, conventional margins of safety in seismic probability Zone 1. Static stability conditions will not be known until a conventional embankment stability analysis, based on data collected by field sampling and laboratory testing, is conducted. Coefficients for seismic acceleration may easily be applied to conventional static load methodology for embankment stability analysis. The absence of faults at the Vaux #1 Dam site indicates that little potential exists for damage to the dam embankment due to the differential movement along foundation faults.

2.4.3.2 Vaux #2 Dam Embankment Stability. The minimum data of unit weight, shear strength, pore pressure, permeability, and embankment zoning, necessary for an embankment stability analysis apparently do not exist for the Vaux #2 Dam and there is no record of a design stability analysis having been conducted.

The original combined 1944 and 1947 embankment fills were failed during the major flood in March, 1951, however, eyewitness accounts of the failure indicate that the overtopped embankments failed by erosion rather than by shear failure.

The presence of springs in the left abutment and an area of obviously high pore pressure in the alluvial foundation materials near the right abutment where "quivering ground" was observed indicate a potential for stability problems. Local residents, including Mr. Sig Anderson, report that the area of quivering ground is a natural spring which was present before the construction of the dam and was relied upon as a source of water supply by homesteaders as early as the 1890's. However, a ground water basin capable of storing and transmitting the amount of discharge observed at the time of the dam safety inspection is not evident anywhere in the area except in the alluvial and outwash sand and gravel filling the Lone Tree Creek valley. It is evident that even if a perennial spring has always existed at the location of the quivering ground, the spring zone is currently transmitting seepage under the Vaux #2 Dam embankment and is a controlling factor in the seepage gradient and the seepage pressure exerted on the dam embankment.

The presence of seepage on the left abutment also raises questions about embankment stability. The path of the seepage is not known. Possible paths for the seepage are through the rolled embankment material, or along the interface between the embankment and the abutment. The rolled embankment should not permit free-flowing seepage, however, compaction near the abutment may have been inadequate due to excessive lift thickness where the embankment gradient changed at the abutment. Similarly, the compaction of keyway material may not have been satisfactory. Photographs taken during the 1960 reconstruction show the embankment and left abutment keyway fill graded to make an access road to the dam during constuction. The abutment material probably consists of Fort Union Formation beds which may include permeable sandstone lenses.

Piping is possible in the more non-plastic zones of either the rolled embankment material or the Fort Union bedrock. The thickness of the outer shell of sand and gravel is unknown at the seepage location. The shell materials are coarse-grained and may not act as a filter to effectively prevent piping. Seepage flows causing piping could transport fine particles into the coarse-grained shell materials. Invasion of the sand and gravel of the outer shell by piped fines might eventually clog the shell material and cause seepage flow perched on the clogged area to discharge on the face of the dam rather than to percolate through the permeable shell materials.

The observed seepage conditions at the Vaux Dam #2 indicate that field investigations are required including installation and monitoring of piezometers, to determine the paths of seepage and the seepage pressure (pore pressures) at several locations in the embankment,

foundation, and abutments. Embankment stability analyses required by the recommended guidelines must consider the seepage conditions in the separate parts of the structure as necessary data. Remarks in section 2.4.3.1 regarding seismic stability and foundation faults apply equally to Vaux #2 Dam.

2.5 PROJECT OPERATIONS AND MAINTENANCE.

The Vaux Dams project is owned by the Lone Tree Ranch Corporation which is solely responsible for the project operation and maintenance. The project is operated as a water storage and regulating facility for agricultural irrigation downstream of the dams. According to the owner, the reservoirs are lowered in early March before the spring runoff and then, if filling occurs, they are again drained some to anticipate further filling which usually comes with June rains. The upper reservoir regulates flow to the lower reservoir which in turn regulates the irrigation releases. The project is generally operated by Mr. Bryce Witt who lives approximately one mile from the lower dam. Mr. Witt visits the dams almost daily during the irrigating season but visits are more sporadic during the other seasons.

2.5.1 Dam Maintenance Plans.

Maintenance is performed on the dams on an as-needed basis such as the repairs that were made to the gate on Vaux #1. The gate at Vaux #2 is also in need of maintenance to correct the binding problem during gate closure. The apron at the Vaux #2 outlet is badly broken and needs to be replaced with proper stabilization to the eroded area around the outlet collar. There is also a heavy growth of willows in the spillway channel which should be removed to maintain maximum spillway capacity. A few trees are growing at the toe of the embankment at Vaux #1 as well as in the abutment contact areas which should be removed. There is no formal maintenance or inspection program which is followed in the operation and maintenance of the dam.

2.5.2 Warning System.

There is no formal warning system for either early detection of high flows or for warning the downstream areas of potentially hazardous conditions at the dams. The owners were familiar with appropriate steps to be taken in the case of emergency conditions and had actually conducted emergency warnings during the dam failures in 1951.

CHAPTER 3

FINDINGS AND RECOMMENDATIONS

3.1 FINDINGS.

Vaux #1 and #2 Dams are two separate earthfill dams located on Lone Tree Creek with each creating its own impoundment. Vaux #2 is the larger of the two dams and is about one half mile upstream of Vaux #1 Dam. The two dams are operated jointly as one irrigation water supply project. The upper dam, Vaux #2, is 46 feet high and impounds 600 A-F of water at normal pool and more than 1,000 A-F at the crest of the dam. Vaux #2 functions as the project's main storage reservoir and also provides total project spillway capacity. The lower dam, Vaux #1, is 24 feet high and impounds approximately 250 A-F at normal pool. Vaux #1 is a storage and regulating reservoir which controls all irrigation releases. A spillway which once served the lower dam has been completely obstructed by the construction of two roads across the channel. Vaux #1 under present conditions has no functional spillway. Almost certainly a failure of Vaux #2 Dam would result in a failure of Vaux #1 Dam. Because of the physical and operational configuration of the dam, Vaux #2 Dam dictates the size classification of the project. As a project, the Vaux Dams are classified as intermediate in size with a high downstream hazard potential.

It is the finding of this Phase I Dam Safety inspection that with the Vaux #2 reservoir at the spillway crest, a flood of approximately 5 percent of the magnitude of the PMF will exceed the spillway capacity and cause the dam to be overtopped. Because an earthfill structure cannot withstand overtopping, Vaux #2 would presumably fail. Vaux #1 Dam, with no functional spillway and less available storage than Vaux #2 Dam, would not be able to handle the flow from Vaux #2 and would also be overtopped and fail.

Failure of the Vaux Dams would present a potential for loss of life and excessive property damage downstream. The recommended spillway design flood (SDF) for this project is a full PMF which has been determined by this preliminary investigation to result from 72-hour June storm with a total precipitation of 23.8 inches. The resulting flood would have a peak inflow of 92,800 cfs with a runoff volume of approximately 77,500 acre-feet. Because the project can safely handle only 5% of the PMF without overtopping and causing the dam to fail, the Vaux dams have a seriously inadequate spillway and are considered unsafe until recommended actions are completed.

Other findings of the inspection address seepage conditions observed at both dams. Seepage was observed on the downstream slope above the toe in the portion of the Vaux #1 embankment containing original 1936 embankment material. Seepage was also observed at Vaux #2 at both abutments. Seepage conditions in the alluvial foundation

near the south abutment and in the embankment at the north abutment may adversely affect embankment stability at higher pool elevations. Observed seepage conditions at both dams suggest that the downstream slopes do not meet minimum stability criteria.

Even under normal operating conditions a situation exists which could possibly cause Vaux #1 reservoir to fail. If for any reason, flow from the outlet works at Vaux #2 were allowed for a long enough period without a corresponding discharge from Vaux #1, with no spillway capacity at Vaux #1 the reservoir would fill and overtop the embankment creating a potentially hazardous condition.

3.2 RECOMMENDATIONS.

3.2.1 Immediately prepare, implement and test an emergency plan for alerting downstream residents in case of imminent dam overtopping or structural destress.

3.2.2 Immediately remove all trees from the Vaux #2 spillway. Also remove all trees, brush and root systems from embankments, for a distance of 100 feet from the abutment contacts, and backfill and compact disturbed areas.

3.2.3 Immediately remove the roads blocking the spillway at Vaux #1 and restore the spillway to service.

3.2.4 Until the Vaux #1 spillway is made operational, prepare and implement a project operating plan to assure the outlet discharge from Vaux #2 Dam does not exceed the discharge and/or storage capacity of Vaux #1 Dam to insure Vaux #1 Dam does not overtop during normal operating conditions.

3.2.5 Prior to the next refill period, accomplish remedial repair to the Vaux #2 Dam spillway channel to arrest the erosion that has occurred about 1300 feet downstream from the entrance.

3.2.6 Repair eroded areas on upstream face of embankments and place adequate riprap protection.

3.2.7 Repair outlet gates to eliminate binding.

3.2.8 Replace the concrete apron at the outlet of Vaux #2 and fill and protect the eroded areas.

3.2.9 Prevent drainage from the road at the south abutment of Vaux #1 from causing erosion of the downstream embankment.

3.2.10 Inspect the outlet conduits throughout their entire length, and, if damaged, repair is needed.

The preceding recommendations (3.2.1 through 3.2.10) will not make the project safe, but will reduce the risk to life and property while the following recommended actions are being taken.

3.2.11 Conduct engineering studies to determine the PMF.

3.2.12 Conduct field and laboratory investigations of foundations and embankment materials, install and read piezometers and perform seepage and stability studies by qualified geotechnical engineers.

3.2.13 As studies indicate, modify the project to safely handle the full PMF, and to provide adequate safety factors for stability of downstream slopes under full pool conditions.

3.2.14 Prepare a program for periodic inspections at least once every five years to detect conditions of significant structural stress and operational inadequacy. This should be a detailed inspection by engineers or other qualified technical experts.

3.2.15 If the project is not made safe within a reasonable period of time, remove the dams in a safe manner to protect downstream life and property.

REFERENCES

1. Alden, W.C., 1932, Physiography and Glacial Geology of Eastern Montana and Adjacent Areas; U.S. Geol. Survey Prof. Paper 174,133 p.

2. Anderson, S.A., Lone Tree Ranch Corporation; Personal Communication, September 27, 1978.

3. Bratner, E.F. and H.W. King, Handbook of Hydraulics, 6th ed., McGraw Hill Inc., 1976, 575 pp.

4. ______ "Conservation News", July 1960; Richland County Soil Conservation District.

5. Driver, Wm. 1971, U.S. Dept. of Agriculture, SCS, Letter to Mel Johnson dated March 5, 1971.

6. U.S. Department of the Army, "Flood Hydrograph Package (HEC-1) - Users Manual for Dam Safety Investigations", Hydrologic Engineering Center, Corps of Engineers, Davis, Calif., 1978, 93 pp.

7. U.S. Department of the Army, Corps of Engineers Hydrologic Engineering Center; "HEC-2 Water Surface Profiles", Generalized Computer Program Users Manual with Supplement; Nov. 76.

8. U.S. Army Corps of Engineers, Office of the Chief of Engineers Report to the U.S. Congress, National Program of Inspection of Dams, Vol. 1, Appendix D, "Recommended Guidelines for Safety Inspection of Dams," Washington, D.C., Department of the Army, May 1975.

9. U.S. Department of Commerce, "Climatological Data Annual Summary - Montana", National Ocean and Atmos. Admin., Asheville, North Carolina, 1977, 21 pp.

10. U.S. Department of Commerce, "Probable Maximum Precipitation Northwest States", Hydrometeorological Report No. 43, U.S. Weather Bureau, Wash., D.C., 1966, 228 pp.

11. U.S. Department of Commerce, NOAA; U.S. Department of the Army, COE; "Probable Maximum Precipitation Estimates, United States East of the 105th Meridian," Hydrometeorological Report No. 51, June 1978, 87 pp.

12. U.S. Department of Interior, "Design of Small Dams", 3rd ed., Bureau of Reclamation, Denver, Colorado, 1977, 816 pp.

PHOTO 3 VAUX #2 DAM AND RESERVOIR

VAUX #2 DAM

INSPECTION PHOTOGRAPHS TAKEN SEPTEMBER 27, 1978

PHOTO 5 VAUX #2 OUTLET CHANNEL LOOKING DOWNSTREAM TOWARD VAUX #1 RESERVOIR

PHOTO 4 VAUX #2, UPSTREAM FACE AND VAUX #1 RESERVOIR

PHOTO 5 VAUX #2 OUTLET CHANNEL LOOKING DOWNSTREAM TOWARD VAUX #1 RESERVOIR

PHOTO 4 VAUX #2, UPSTREAM FACE AND VAUX #1 RESERVOIR

PHOTO 6 VAUX #2 FROM NORTH ABUTMENT

PHOTO 7 VAUX #2 UPSTREAM FACE WAVE EROSION

PHOTO 8 VAUX #2 DOWNSTREAM FACE

PHOTO 9 VAUX #2 SOUTH ABUTMENT DOWNSTREAM FACE

PHOTO 10 VAUX #2 SEEPAGE AREA AT TOE OF SOUTH ABUTMENT TAKEN FROM DAM

PHOTO 11 VAUX #2 NORTH ABUTMENT AND OUTLET

PHOTO 12
VAUX #2 GATE CONTROL

PHOTO 13
VAUX #2 GATE SHAFT
WET WELL

PHOTO 14
VAUX #2 OUTLET CONDUIT
AND APRON

PHOTO 15 VAUX #2 SPILLWAY INLET — LOOKING UPSTREAM

PHOTO 12
VAUX #2 GATE CONTROL

PHOTO 13
VAUX #2 GATE SHAFT
WET WELL

PHOTO 14
VAUX #2 OUTLET CONDUIT
AND APRON

PHOTO 15 VAUX #2 SPILLWAY INLET — LOOKING UPSTREAM

PHOTO 17 ERODED SECTION OF SPILLWAY APPROXIMATELY 1300 ft. DOWNSTREAM OF INLET

PHOTO 16 VAUX #2 SPILLWAY INLET AND CHANNEL LOOKING DOWNSTREAM

PHOTO 18 ERODED SECTION OF SPILLWAY — LOOKING UPSTREAM

PHOTO 19 OUTLET OF SPILLWAY CHANNEL AT BRORSON CREEK

PHOTO 20 GLACIAL TILL IN SPILLWAY

PHOTO 21 VAUX #2 ESKER STANDING ABOVE OUTWASH PLAIN NORTH OF RESERVOIR

PHOTO 22 VAUX #1 RESERVOIR

VAUX #1 DAM

INSPECTION PHOTOGRAPHS TAKEN SEPTEMBER 27, 1978

PHOTO 23 VAUX #1 DAM, UPSTREAM FACE

PHOTO 24 VAUX #1 DAM CREST AND DOWNSTREAM FACE FROM SOUTH ABUTMENT

PHOTO 25 VAUX #1 WETTING ZONE ON FACE OF DAM NORTH OF OUTLET

PHOTO 24 VAUX #1 DAM CREST AND DOWNSTREAM FACE FROM SOUTH ABUTMENT

PHOTO 25 VAUX #1 WETTING ZONE ON FACE OF DAM NORTH OF OUTLET

PHOTO 26 VAUX #1 SOUTH ABUTMENT AND GATE CONTROL

PHOTO 27 VAUX #1 SURFACE DRAINAGE AT SOUTH ABUTMENT

PHOTO 28 VAUX #1 UPSTREAM FACE AT SOUTH ABUTMENT. NOTE: SLUMP AREA

PHOTO 29 VAUX #1 HIGH WATER AND WAVE EROSION ON UPSTREAM FACE TAKEN FROM SOUTH ABUTMENT

PHOTO 30 VAUX #1 OUTLET AND DOWNSTREAM CHANNEL NEAR SOUTH ABUTMENT

PHOTO 31 VAUX #1 NORTH ABUTMENT AT CONTACT WITH NATURAL GROUND — TAKEN FROM TOP OF DAM

PHOTO 32 VAUX #1 ABANDONED SPILLWAY AT NORTH ABUTMENT

PHOTO 34 VAUX #1 LIP OF ABANDONED SPILLWAY AT DROP TO BRORSON CREEK

PHOTO 33 VAUX #1 ABANDONED SPILLWAY LOOKING DOWNSTREAM

PHOTO 35 VAUX #1 SLIDE GATE CONTROL AND ENTRANCE TO GATE SHAFT

PHOTO 36 VAUX #1 OUTLET AND DIVERSION STRUCTURE

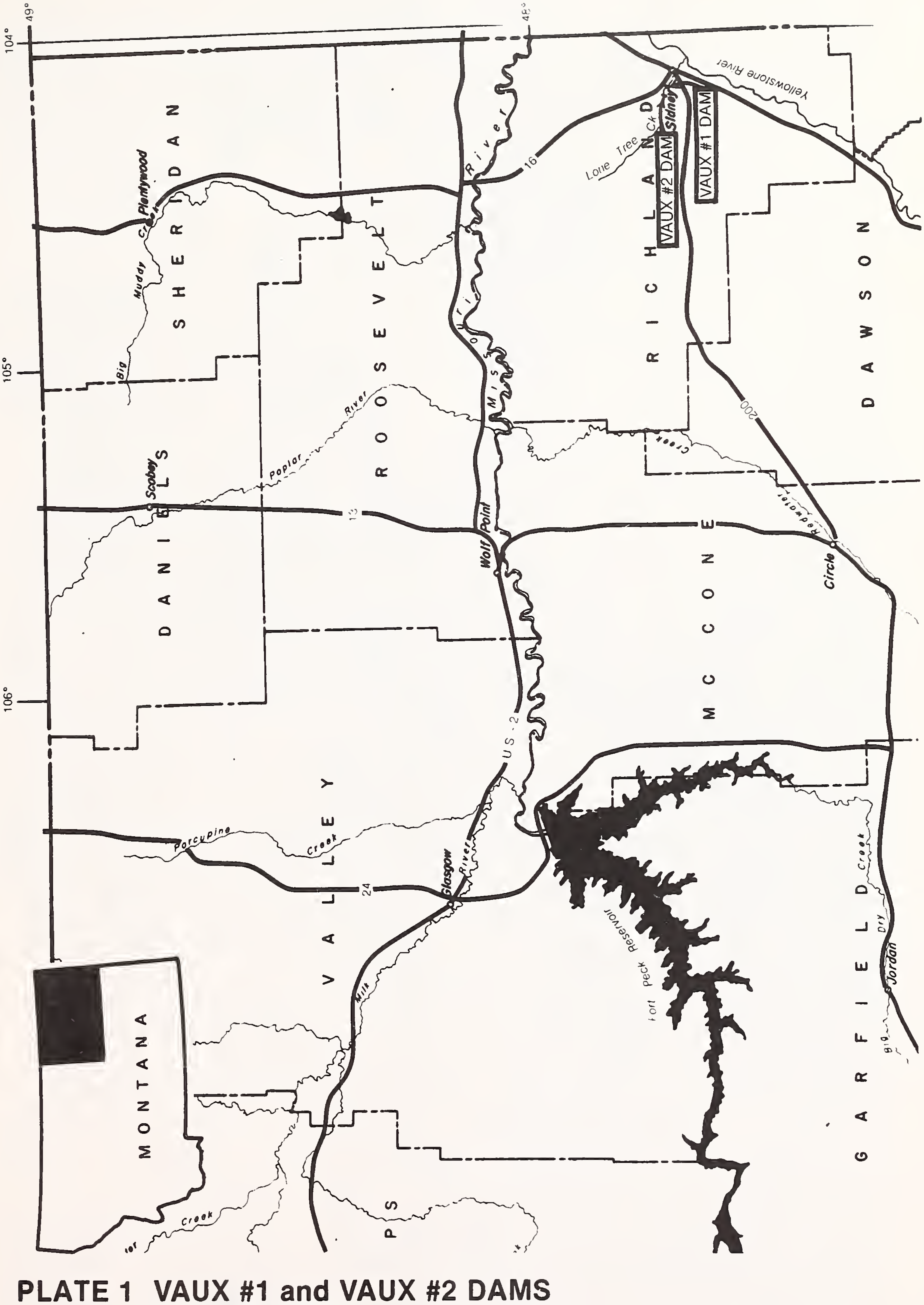


PLATE 1 VAUX #1 and VAUX #2 DAMS VICINITY MAP

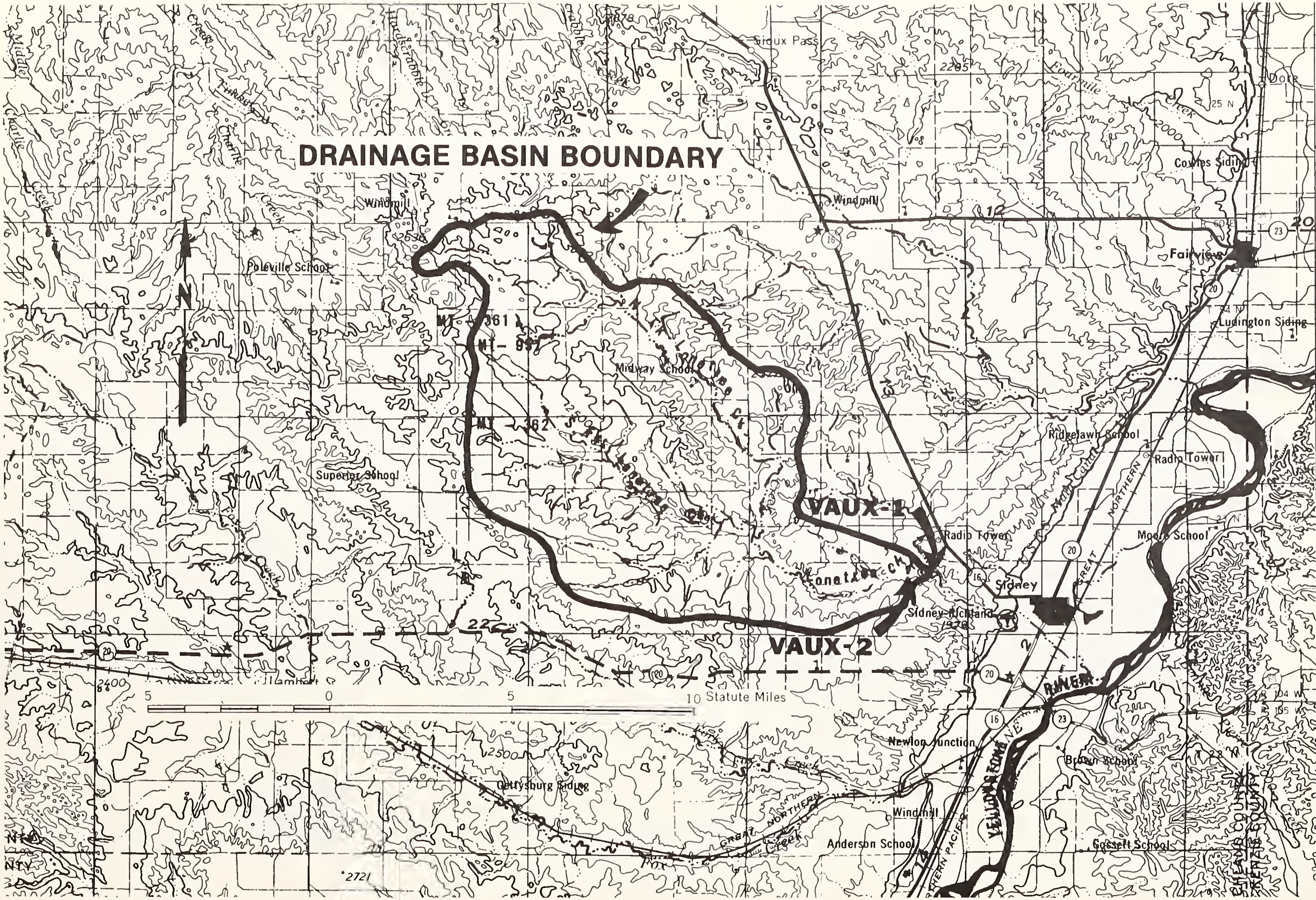


PLATE 2 DRAINAGE BASIN MAP

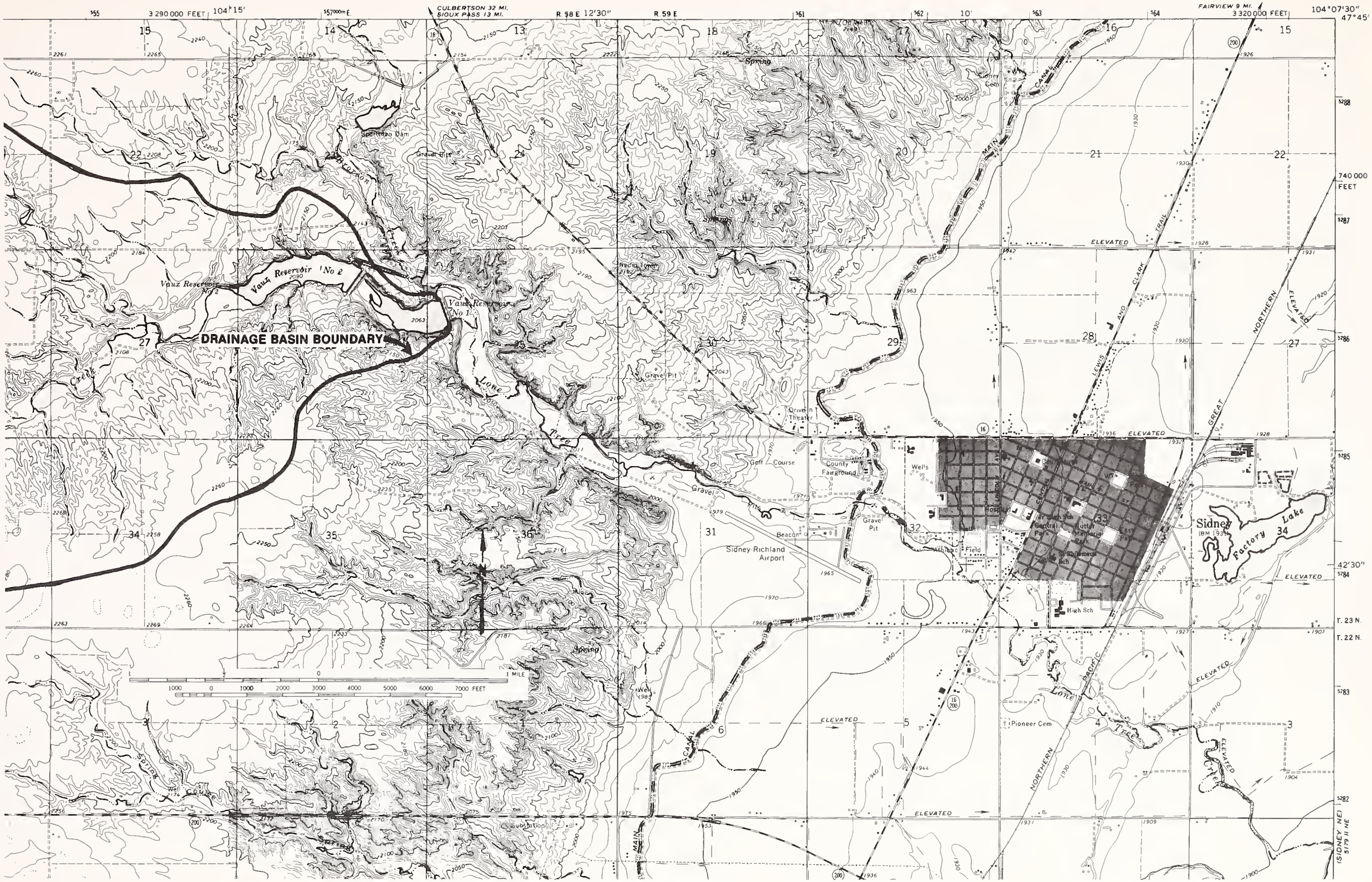
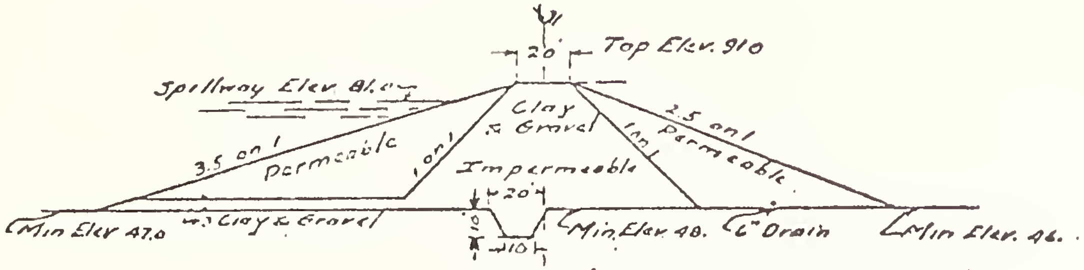


PLATE 3 DAM SITE MAP

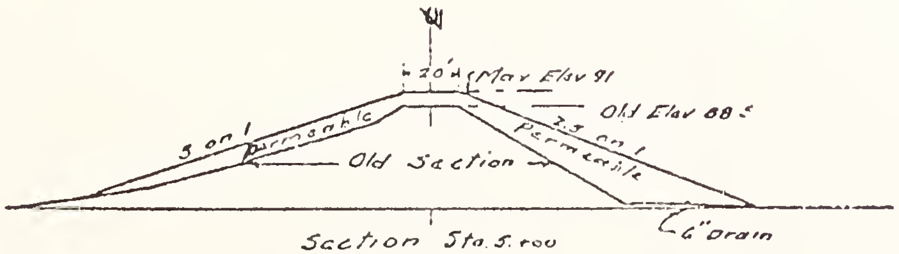
PLATE 4

DESIGN UPPER DAM



Typical
Rebuilt X-Section Sta. 7+50 to 10+25

VAUX #2

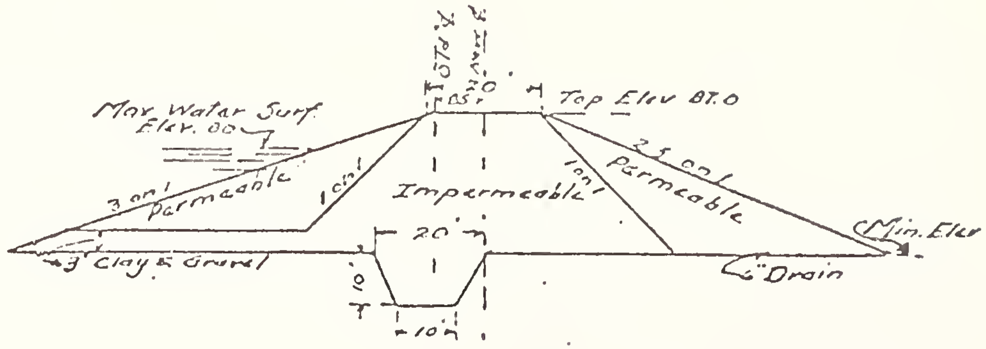


Typical
Repaired Section Sta. 0+75 to 7+50

PLAN FOR REPAIR
S. A. ANDERSON DAM
NEAR SIDNEY MONT.

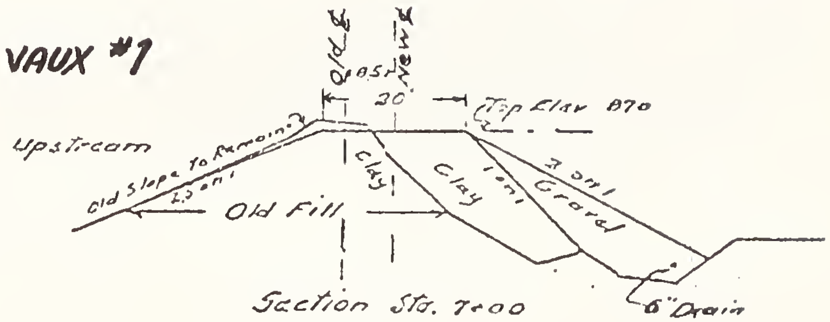
LILLIS ENGINEERING
Jan. 1954

DESIGN FOR LOWER DAM



Typical
Rebuilt X-Section Sta 0+70 to 5+20

VAUX #1



Typical.
Repaired Section Sta. 5+20 to 9+00

PLAN FOR REPAIR
S.A. ANDERSON DAM
NEAR SIDNEY MONT.

LILLIS ENGINEERING
Jan. 1954.

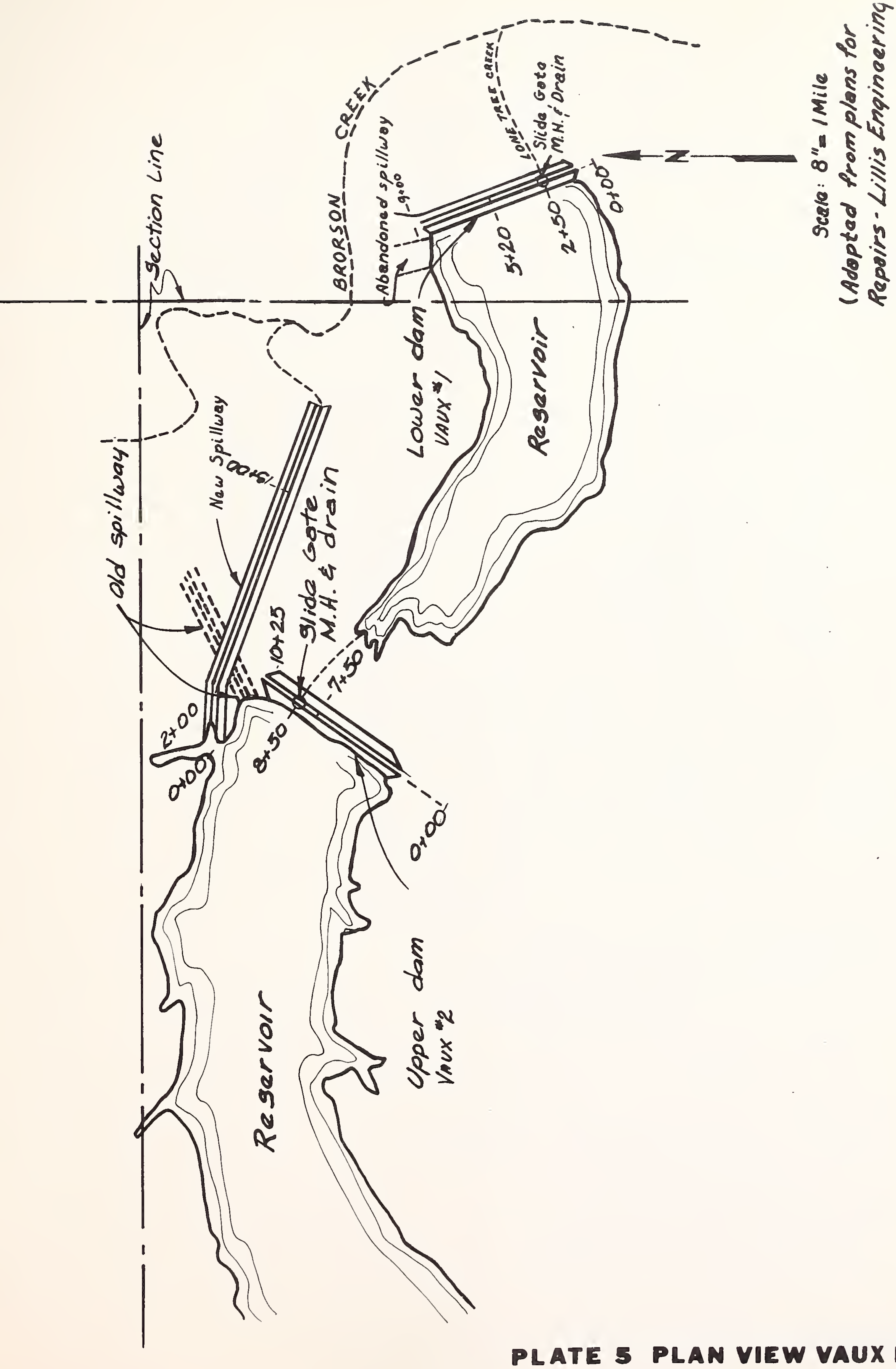
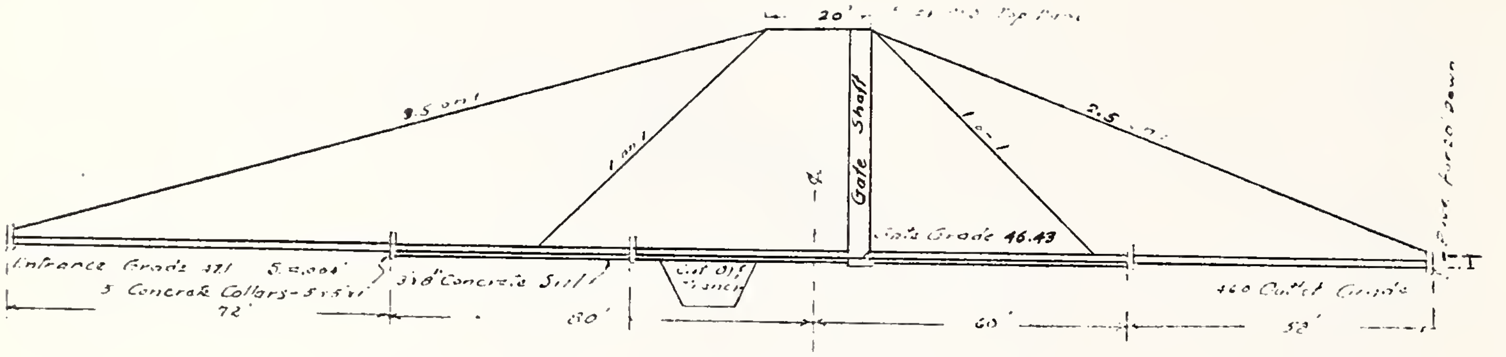
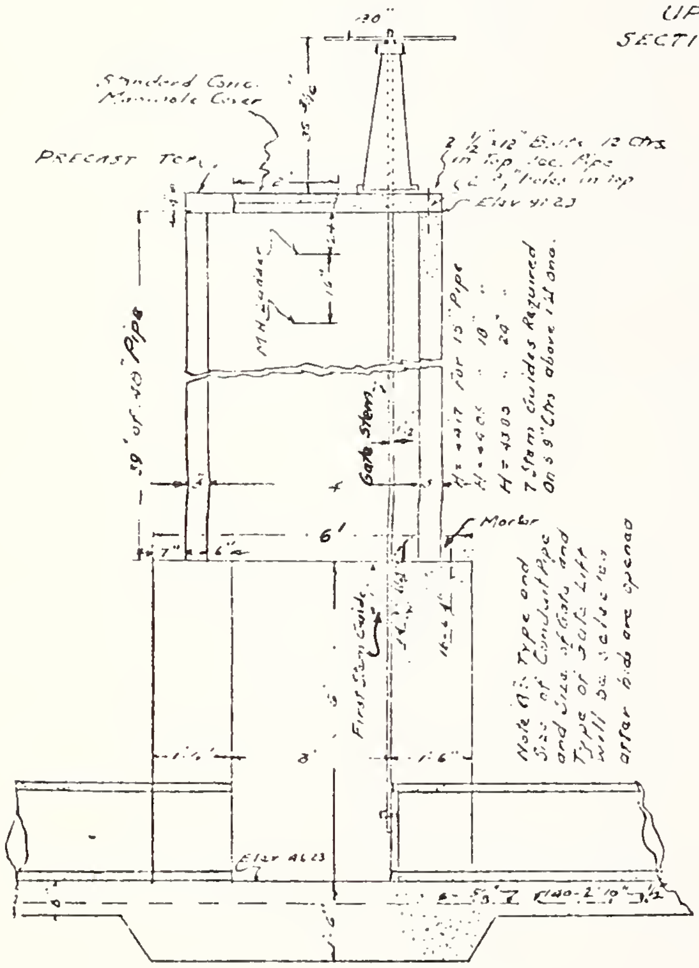


PLATE 5 PLAN VIEW VAUX DAMS



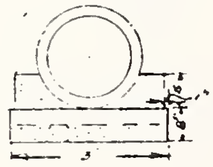
UPPER DAM
SECTION AT STA 8+30



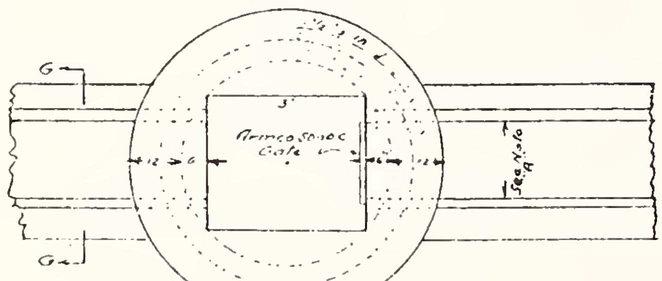
SECTION OF GATE SHAFT

REINFORCING LIST

Location	Size	No.	Length
Pipe Foundation			
Longitudinal	5/8"	10	40'
"	"	6	26'
Transverse	1/2	140	2'-10
Shaft Foundation			
Circ Ins Horizontal	1/2	6	16'
" Outs "	"	6	18"
Vert. Ins	"	14	6'-4"
" Outs	"	16	6'-8"
Shaft Top Cover			
Circular	"	1	16'
Transv	5/8	1	3'
"	"	2	4'
Longi	"	2	4'-6
"	"	1	4'
Pipe Collars Vert	1/2	415	4'-6"
Hori	"	415	4'-6"
Total Length of 5/8"		504'	
" Weight " "		913 lbs.	
" Length " 1/2"		995'	
" Weight " "		665 lbs.	
Grand Total Weight		1050 lbs.	



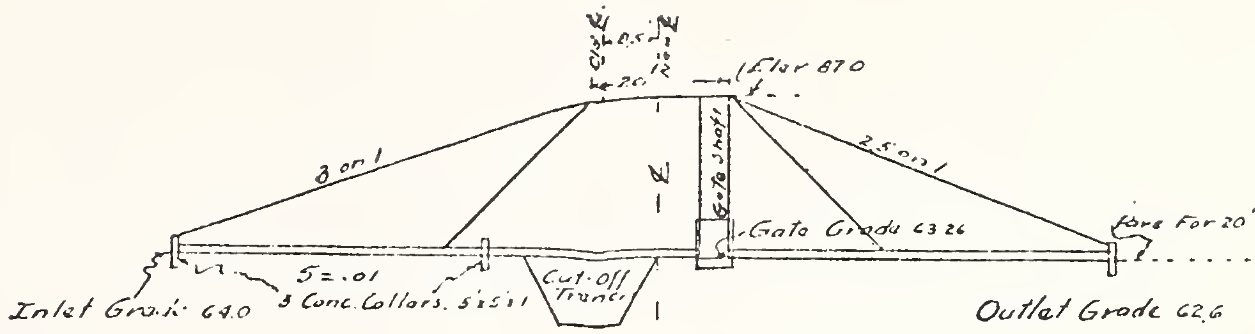
SECTION G.G.
EXTENDING 80' ABOVE AND 60' BELOW ℄ OF DAM



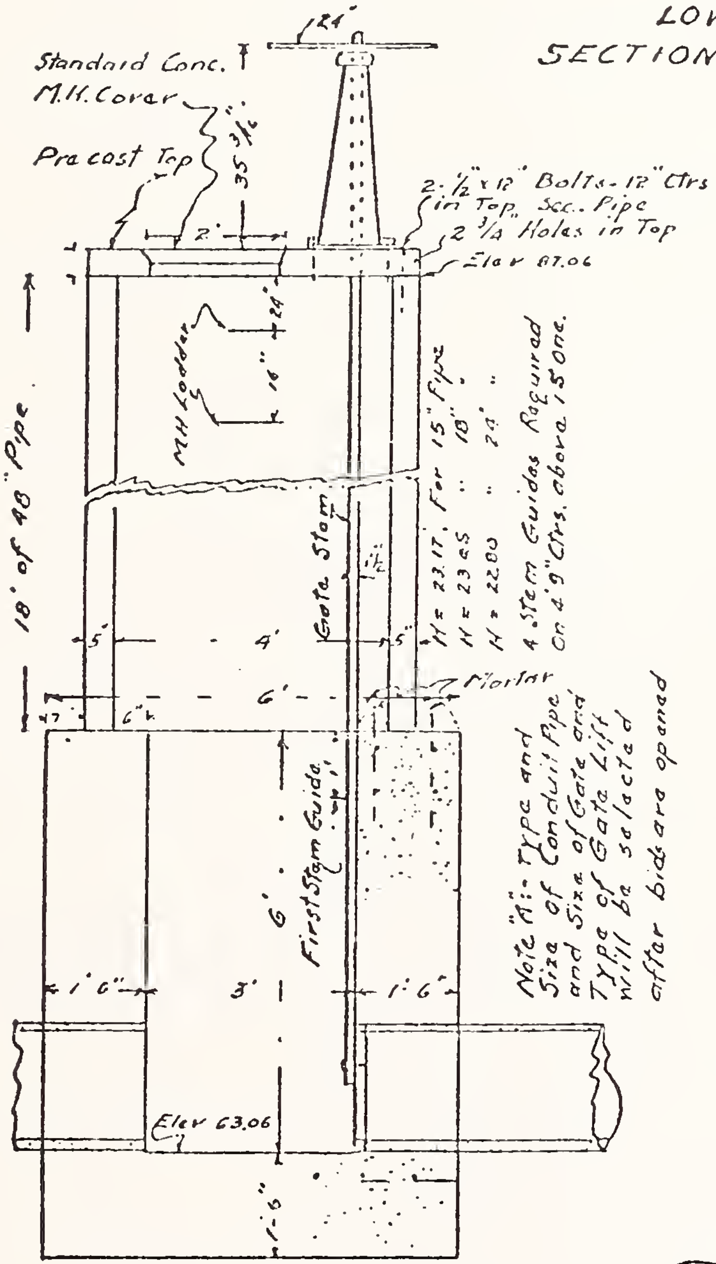
GATE SHAFT PLAN

S.A. ANDERSON DAMS.
PLANS FOR REPAIRS
UPPER DAM
CONDUIT AND GATE SHAFT
Jan. 1954 Lillis Engineering

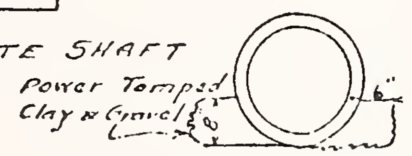
PLATE 6



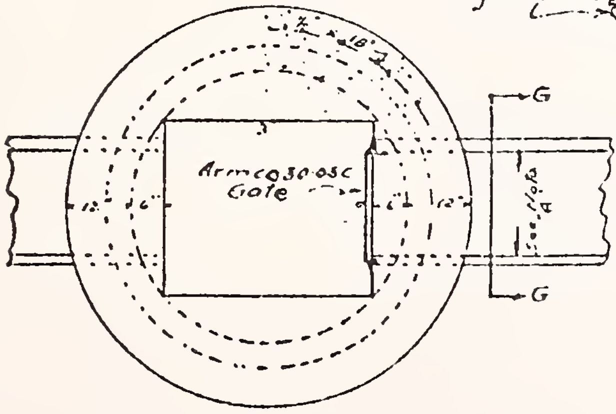
LOWER DAM
SECTION AT STA. 2+50



SECTION OF GATE SHAFT



SECTION G.G.



GATE SHAFT PLAN

REINFORCING LIST

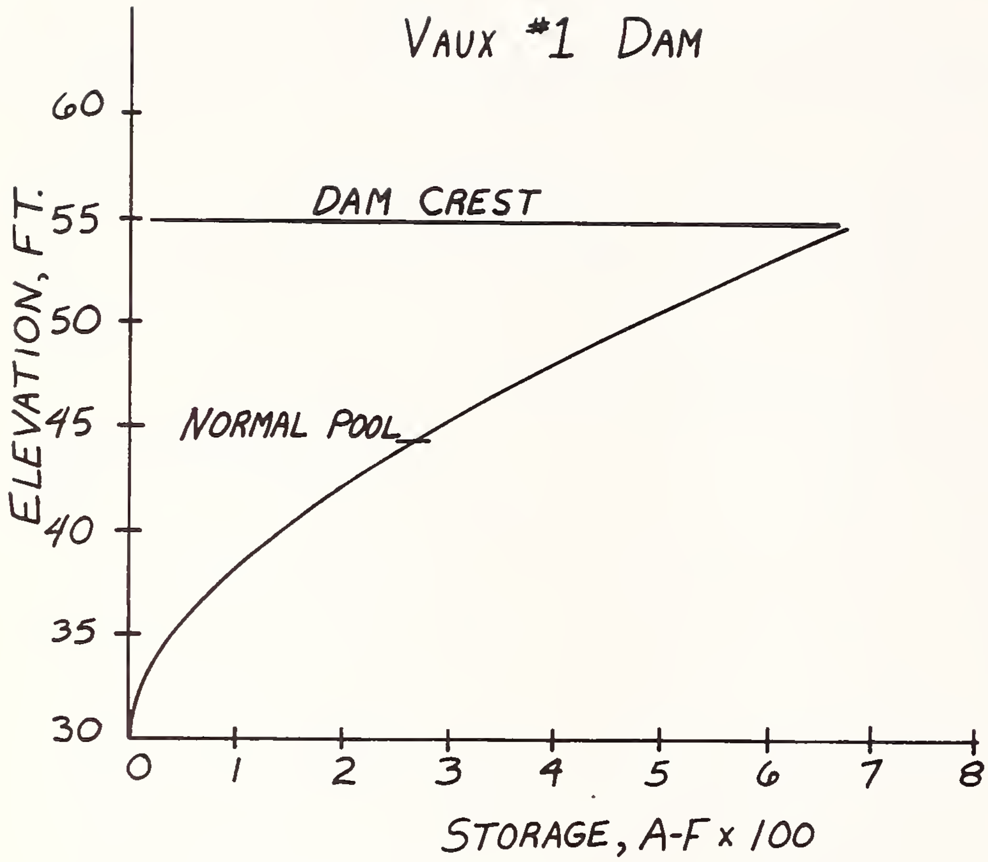
Location	Size	No.	Length
Shaft Foundation			
Circ. Ins. Hori	½"	6	16'
" Outside "	"	6	18'
Longitudinal	5/8	6	6'
Transverse	"	6	6'
Vert. Inside Row	½"	14	6'-4"
" Outside "	½"	16	6'-4"
Shaft Top Cover.			
Circular	½"	1	16'
Transv.	5/8	1	3'
"	"	2	4'
Longi.	"	2	4'6"
"	"	1	4'
Pipe Collars Vert.	½"	4x3	4'6"
Hori.	"	4x3	4'6"
Total Length of 5/8" ϕ		96'	
" Weight "		100. lbs.	
" Length " ½" ϕ		521'	
" Weight " "		350 lbs	
Grand Total Weight		450 lbs.	

NOTE: SUBTRACT 33.4 FEET TO MATCH VAUX #2 DATUM

S.A. ANDERSON DAMS.
PLANS FOR REPAIRS
LOWER DAM
CONDUIT AND GATE SHAFT

Jan. 1954. Lillis Engineering

PLATE 7



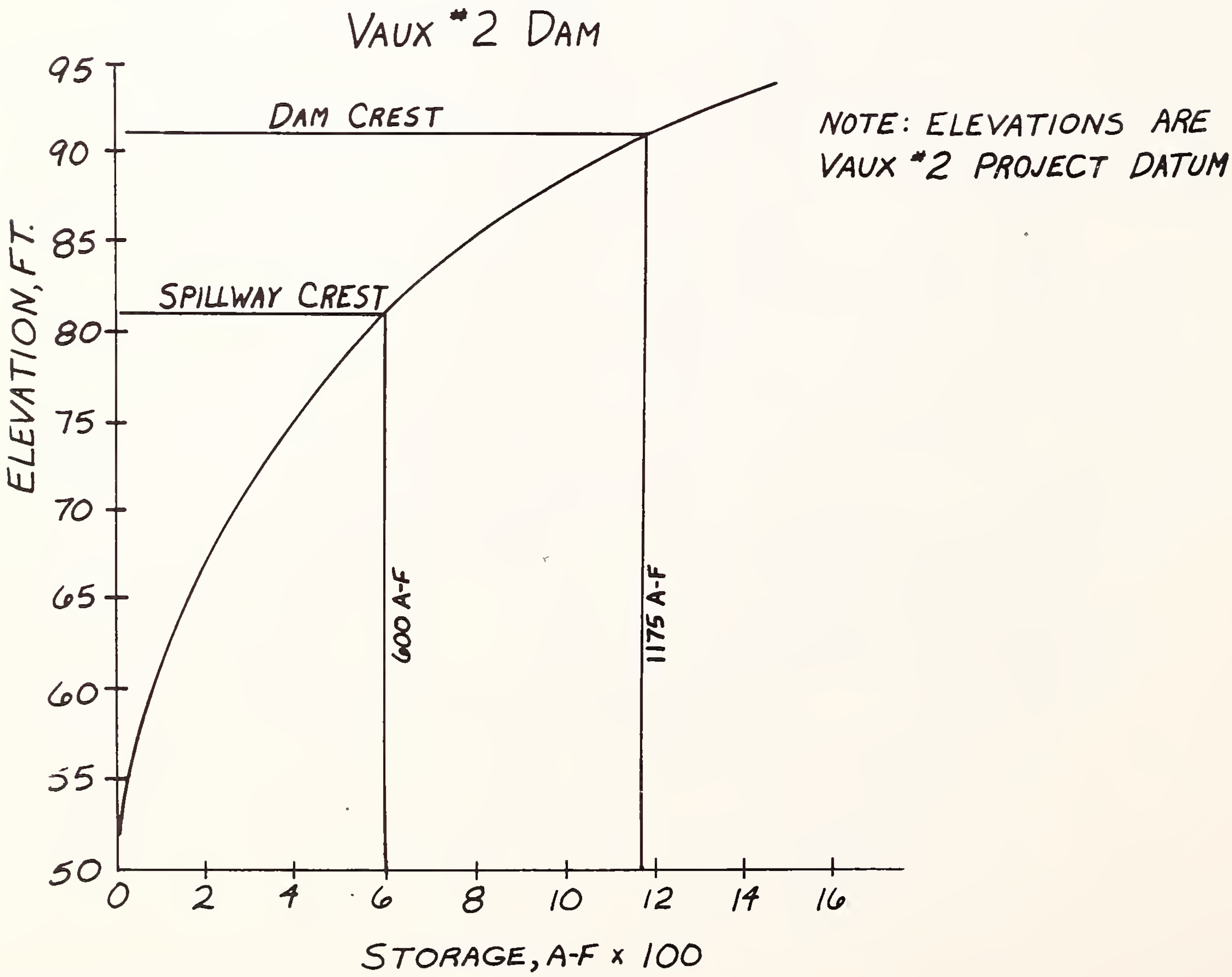
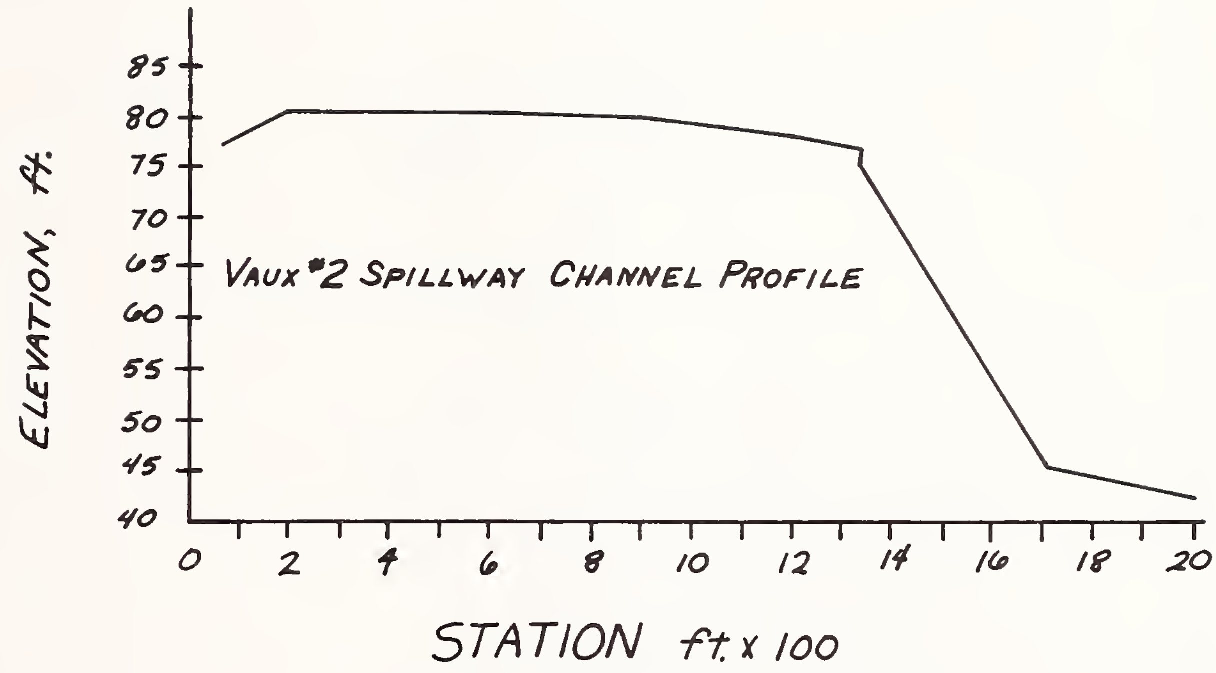


PLATE 8 ELEVATION — STORAGE CURVE



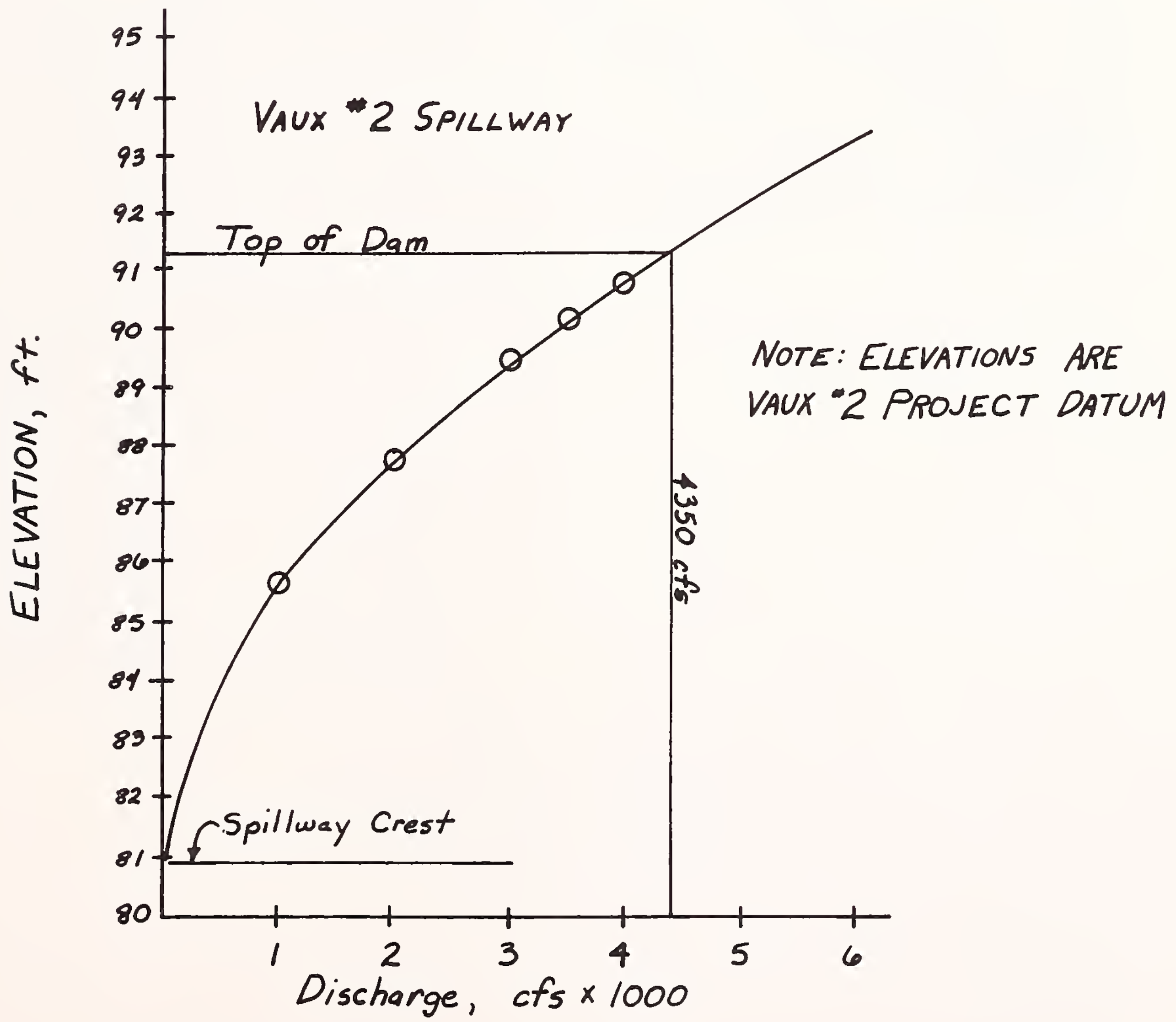
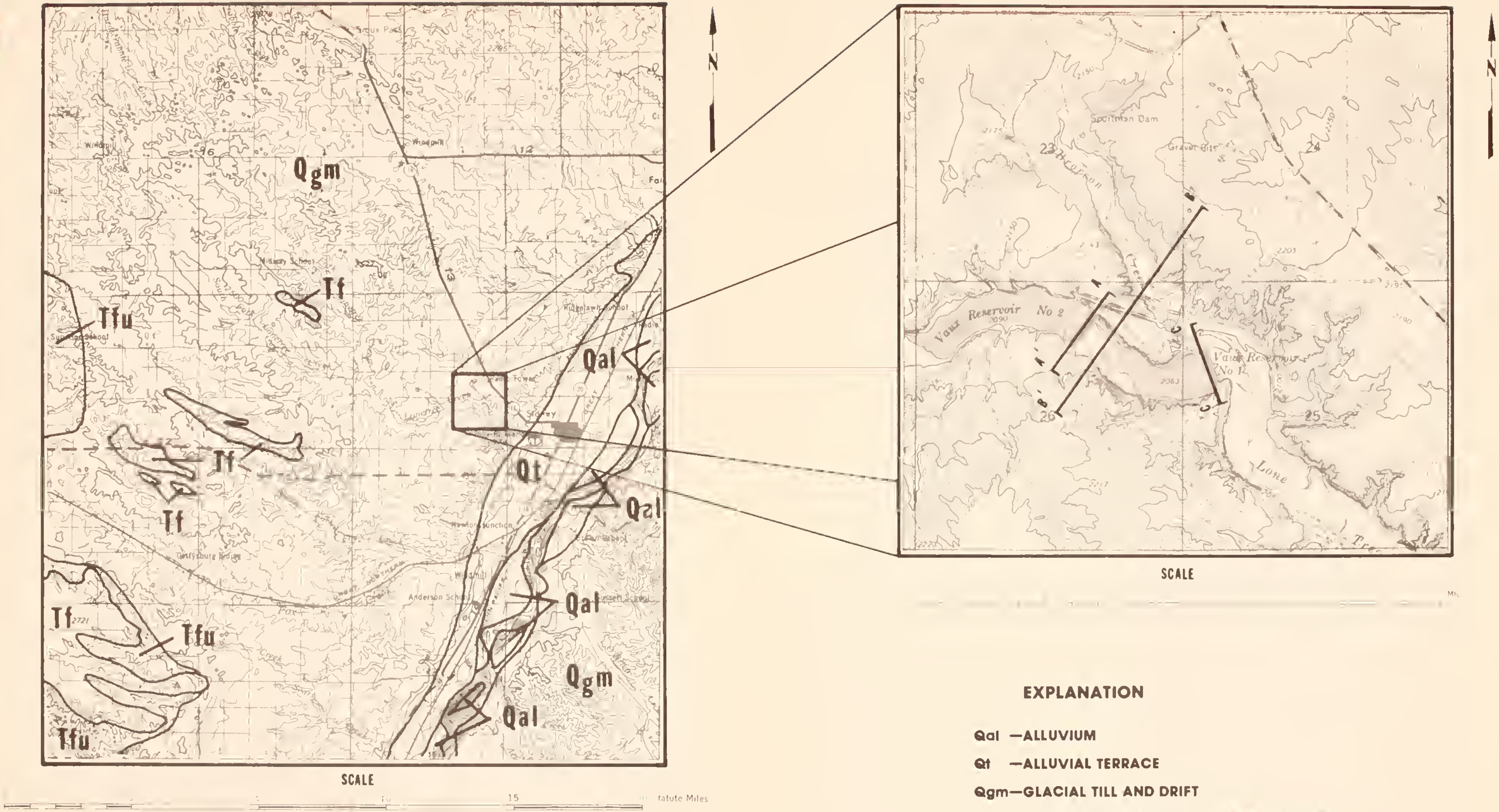


PLATE 9 DISCHARGE RATING CURVE



BASED ON ALDEN (1932)

EXPLANATION

Qal —ALLUVIUM

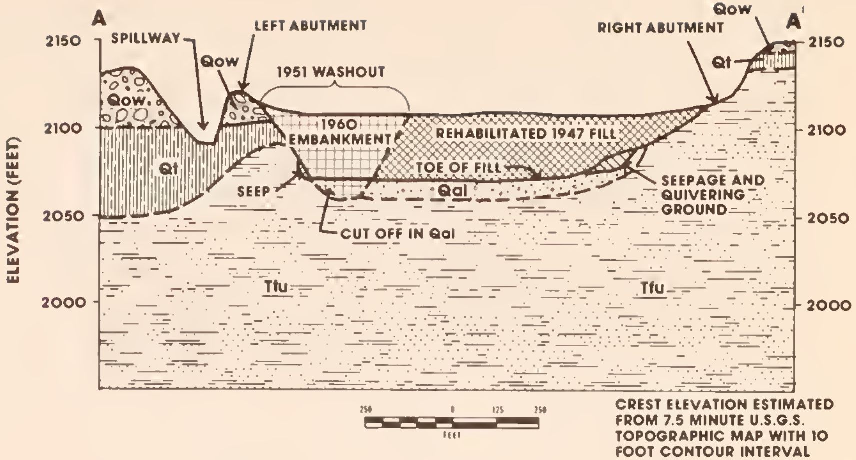
Qt —ALLUVIAL TERRACE

Qgm—GLACIAL TILL AND DRIFT

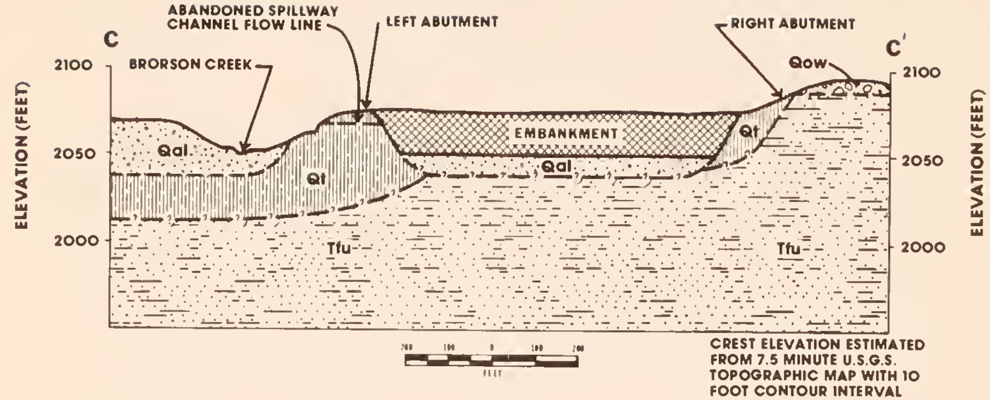
Tf —FLAXVILLE TERRACE OR PROBABLE EQUIVALENT

Tfu —FORT UNION FORMATION

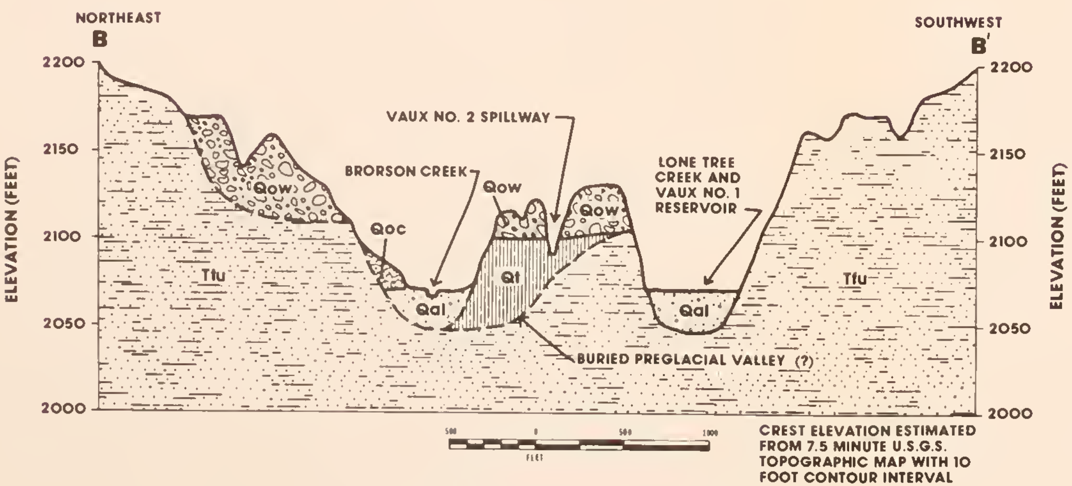
PLATE 10 GENERAL GEOLOGIC MAP



SCHEMATIC GEOLOGIC CROSS SECTION OF VAUX DAM NO. 2



SCHEMATIC GEOLOGIC CROSS SECTION OF VAUX DAM NO. 1



SCHEMATIC GEOLOGIC CROSS SECTION

EXPLANATION

Qal —ALLUVIUM

Qac —COLLUVIAL SCOPE WASH DEPOSIT

Qow—GLACIAL OUTWASH GRAVEL

Qt —GLACIAL TILL

Tfu —FORT UNION FORMATION

—— —IDENTIFIED GEOLOGIC CONTACT

— — —EXTRAPOLATED OR ESTIMATED GEOLOGIC CONTACT

—?— —UNCERTAIN OR HYPOTHESISED GEOLOGIC CONTACT

PLATE 11 GEOLOGIC CROSS SECTIONS

CORRESPONDENCE

Lone Tree Ranch inc.
SIDNEY, MONTANA 59270
Box 472

July 31, 1979

Department of the Army
Seattle District, Corps of Engineers
P. O. Box C-3755
Seattle, Washington 98124

Gentlemen:

We received your draft of the report on Vaux #1 and #2 dams on July 14, 1979, and a request to reply and comment by July 23, 1979 in your enclosure letter. The time you allowed in which to reply seems a bit irresponsible since you took ten months to develop the report.

In response to your recommendation we feel we have already demonstrated that we have responsibly warned the public of danger in the 1951 flood which was not the result of engineering or structural design failure of the dam or spillways but from the negligent engineering design of a bridge by the Richland County Road Department in the spillway which obstructed ice flow and which collapsed and effectively made the spillway nonfunctional. When the spillway was initially constructed in 1947 a county road was abandoned and several years later in response to a request to reopen the road an agreement was made by Mr. Vaux and the County Commissioners that grades would be made to the floor of the spillway. For unknown reasons the County built a bridge instead, unfortunately we did not object, assuming incorrectly that the County Road Department knew what it was doing.

Your second recommendation seems irrelevant since you state that inflow is not significant except for failure of Vaux #2. The road that is higher than the crest of Dam #1 is a road built by Cenex for access to a drilling site that is to be abandoned. The road will be lowered after pipe has been removed and the hole plugged. The crest of the other road is several feet lower than the dam crest and will wash out in the event of an accidental overfilling of the dam via the Vaux #2 outlet.

Repairs of the spillway and outlet to Vaux #2 have been in progress over the last month. We have also installed an irrigation system which will greatly increase the draw down of the #2 reservoir and this will enhance the flood protection already provided by the dams.

We have contracted with Thomas, Dean & Hoskins, Inc. to review your report and we are appending their report which we wish to have included in the final report. We feel the dams are structurally sound but agree it is prudent to do test drilling and pieziometer checks to confirm and monitor our beliefs. Contracts for the studies will be made.

We find your PMF for the area to be absurd unless you have knowledge of an impending event as reported in Chapter 6 of Genesis. We would like to know on what basis you determined the PMF for the area as it has no correlation to factual data which is available and on what any prudent engineer would base his design, and as was done Mr. Lillis.

We have always had and executed the coordination of inflow and outflow of the Vaux #1 reservoir by the ranch operator via the control gates and cannot see how writing on a piece of paper will guarantee the operation as much as the physical action of controlling the gates as has been demonstrated by 19 years of safe operation.

This spring's runoff was probably the greatest ever experienced since the construction of the dams and the spillway easily performed its function. The flood plain near Sidney has been protected to a great extent by these dams and the flooding experienced this spring by buildings constructed in the historical flood plain of the creek was from the flooding of the entire creek drainage. It might be wise for the C of E to warn the public of the dangers of building in what is the natural flood plain of the creek, which is a much more likely event than dam failures.

Other minor errors in the narrative were noted on the bottom of page 18, the gate has never been a problem on Vaux #2. On page 34 (2.5), the reservoirs are not filled as early as possible, they are lowered in early March before the spring runoff and then if filling occurs they are again drained some to anticipate further filling which usually comes with the June rains. On page 4, the operator is Mr. Bryce Witt.

Sincerely yours,

Hugh V. Anderson, M.D.
President
Lone Tree Ranch, Inc.

encl

REVIEW OF THE

PHASE I INSPECTION REPORT

NATIONAL DAM PROGRAM

FOR THE

LONE TREE CREEK BASIN

VAUX #1 AND VAUX #2 DAMS

SIDNEY, MONTANA

MT-357 AND MT-358

JULY, 1979

THOMAS, DEAN & HOSKINS, INC.
ENGINEERS
GREAT FALLS - BOZEMAN - KALISPELL
MONTANA

REVIEW
OF THE
PHASE I INSPECTION REPORT
NATIONAL DAM PROGRAM
FOR THE
LONE TREE CREEK BASIN
VAUX #1 AND VAUX #2 DAMS
SIDNEY, MONTANA

MT-357 AND MT-358

COMMENTS ON HYDROLOGY

The report indicates the probable maximum flood entering the upstream reservoir (Vaux #2) is 93,000 cfs. The total drainage area is 83 sq. miles. A flood flow of 93,000 cfs. seems extremely unlikely from such a small drainage area particularly when compared to floods on very large river basins. U.S.G.S. Water Supply Paper 1679 gives maximum floods on various rivers in the Missouri River Basin from 1911 to 1963. The Yellowstone River at Sidney has a drainage area of 68,812 sq. miles and is uncontrolled. The maximum annual Yellowstone flood flow at Sidney exceeded 100,000 cfs on only 11 occasions in 52 years and had an all time peak flow of 159,000 cfs during that period. The average annual flow in the Yellowstone at Sidney is about 10,800 cfs. The river is very large at this point. Another example is the Missouri River. The average Missouri River flow at Culbertson, Montana is 9,760 cfs and is a river of substantial size. Since the river is controlled by dams, flood data at Culbertson is relatively meaningless but the Missouri above Toston, Montana is essentially uncontrolled with a drainage area of 14,669 sq. miles and has a maximum

recorded flood of 32,000 cfs with an average flow of 5,224 cfs. This data is presented to give a relative feeling for the volume of water represented by 93,000 cfs. We do not feel a probable maximum flood flow of this magnitude is reasonable for the drainage area above the dams in question.

A check was made using a flood prediction method developed by Dr. E. R. Dodge of Montana State University for the Montana Department of Highways. The method is based on parametric equations developed from statistical analysis of 230 watersheds in the State of Montana. Sidney lies in an area designated as Region V - Central East. The 100 year flood for an 83 sq. miles area with less than one percent forest cover in this region is predicted at 21,340 cfs. We do not know the relationship of the 100 year flood to what is termed the probable maximum flood as defined by governmental agencies.

Another check was made to determine the precipitation probability for the month of June and the total annual precipitation. A probability analysis was made on 23 years of weather data available from the Fairview recording station. There is a one percent probability (1 chance in 100) that the total annual precipitation will exceed 26 inches. There is also one chance in 100 that the total precipitation for the month of June will exceed 8.5 inches. June is the highest precipitation month.

The report used a maximum precipitation rate of 23.8 inches in a 72 hour period in June. This seems excessively high in view of the probability analysis of existing data. By assuming

the total projected monthly 100 year precipitation of 8.5 inches falls in 3 days and that the resultant flood is proportional to the flood data presented in the report, the peak projected flood would be $\frac{8.5}{23.8}$ x 93,000 or 33,200 cfs. This also seems like ~~a very conservative and~~ GAH an extremely high flow from such a small drainage basin.

A potential precipitation rate of 23.8 inches in 72 hours is incredibly high for this climate. If this precipitation rate did occur, these two small dams would be of minor importance in the total flooding in the area. The dams have a maximum over-topping storage capacity of approximately 1,860 acre-feet. This represents 14.5 minutes of flow at 93,000 cfs. If the dams failed under these conditions, their contribution to the total flood would be negligible.

We feel that agreement must be made on a reasonable definition of the probable maximum flood. The hydrology methods utilized in this report do not appear to provide realistic potential flood values. Any improvements required by governmental agencies should be based on more realistic flood values.

COMMENTS ON DAM EMBANKMENT STABILITY

The Corps report states that the stability of the Vaux #1 and Vaux #2 dam embankments are questionable. The minimum data of unit weight, shear strength, permeability and embankment zoning necessary for an embankment stability analysis apparently do not exist for either dam and there is no record of a design stability analysis ever having been conducted.

Seepage conditions were observed at both dams on the downstream embankment. A quick condition was observed in the vicinity of the right abutment of the Vaux #2 dam. Phreatic seepage and free flowing seepage was observed on Vaux #1 dam. The Corps based its "questionable stability" classification on the observed seepage and its location. There are also wave cut benches on the upstream embankments of both dams.

The Corps report further states that there was no evidence of differential settlement, slope failure, or misalignment in the embankments. Flow from the seepage areas are clear and there is no evidence of piping or accelerated erosion. Further, the dams have existed in their present condition for approximately 19 years (dam rebuilt in 1960).

In view of the fact that the dams are classified as category 1 - high downstream hazard potential, I feel that a complete stability analysis is warranted. This could be accomplished by field investigations, soil borings, installation and monitoring of piezometers, data compilation and analysis.

DEPARTMENT OF NATURAL RESOURCES & CONSERVATION

WATER RESOURCES DIVISION

STATE OF MONTANA

32 SOUTH EWING
HELENA, MT 59601

THOMAS L. JUDGE, GOVERNOR
TED J DONEY, DIRECTOR

RECEIVED

August 16, 1979

Ralph Morrison
Department of the Army
Seattle District - Corps of Engineers
P. O. Box C-3755
Seattle, Washington 98124

Dear Mr. Morrison:

The Department of Natural Resources and Conservation has reviewed the final draft report on Vaux No. 1 and No. 2 dams. We feel the report is complete and satisfies the criteria for the Phase I evaluation.

Thank you for this opportunity to review and comment on the final draft report for the Vaux No. 1 and No. 2 dams.

Sincerely,

Richard L. Bondy

Richard L. Bondy, Chief
Engineering Bureau
(406) 449-2864

RLB/lh